

CONCRETE AND CONSTRUCTIONAL ENGINEERING

OCTOBER, 1933.



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EDITORIAL NOTES.

Cotton Fabric in Concrete Road Construction.

A METHOD of road construction has been developed in the United States in which the two courses of concrete are separated by cotton or linen fabric. The bottom course is a relatively weak mixture; upon this, before the concrete has set, the fabric is laid and tamped sufficiently to permit of mortar passing through the openings in the fabric and incorporating the latter in the concrete. The top course is a rich mixture containing small aggregate, and is laid immediately after the placing of the fabric.

Experiments have been carried out by the Ministry of Transport's Technical Advisory Committee to determine whether difficulties are encountered during construction, and to investigate the claim made for the method that good adhesion can be obtained between the two courses, and that the top course can be removed and replaced without damage to the bottom course. Six test slabs, each 10 ft. square, were laid in the grounds of the Experimental Station, Harmondsworth, where they were not subjected to traffic. The results are published in "Experimental Work on Roads" (London: H.M. Stationery Office. Price 1s. 6d. net). The best bond was obtained with open mesh material, but wherever fabric was used it was easy to separate the courses, and the bond was weaker than in the two-course slab without fabric. The tests do not indicate how the top courses would behave if they were to carry traffic, but the general result of the experiment is to show that the use of cotton and linen fabrics in the manner described is practicable and likely to attain the objects claimed. It does not seem, however, that any practical gain, economic or otherwise, is likely to result from the use of this method in the normal practice of concrete road construction, since experience in this country indicates that concrete roads rarely fail as a result of abrasion.

From the conclusion just quoted from the Report of the Ministry's Committee it would appear that the real object of adopting this type of concrete road has been overlooked. Fabric is not used to enable the wearing surface of slabs which are badly abraded to be replaced—abrasion, as the Report states, rarely causes failure. When fabric is inserted between the slabs the object is to facilitate the replacement of cracked portions. Experiments have been made for several years at a London works' entrance where the roadway was laid as two-course concrete with an intervening layer of fabric and is continuously subjected to heavy lorry traffic. At these works sections of the wearing surface about 1 sq. yd. in area have been cut out after 12 or 18 months' wear, and have

been replaced. It has been found that the surfacing is easily removed without pneumatic tools, this operation being little more difficult than the removal of an asphalt coating. The concrete can also be replaced so that it is scarcely distinguishable from the original surface.

We understand that this method of construction is now being adopted on a main road in the London area, and that it has been applied on the embankment leading to a new bridge in South Wales. In the latter case it is intended to remove the top course when the filling has settled and replace the concrete by a greater thickness which will raise the level of the road surface to that of the carriageway on the bridge, thus reducing the cost of the adjustment that is always required in such cases.

Piling at Cherbourg Marine Station.

ON July 30 the President of the French Republic opened the new harbour works at Cherbourg, including the new railway station which had just been completed. The cost of the latter is approximately 69,000,000 frs., this figure including the costs of the reinforced concrete, foundations, roofing and walling, cranes and other mechanical equipment, roads, water mains, rail tracks and junctions connecting the new station to the existing lines. The cost of the reinforced concrete, foundations, and drainage was approximately 15,000,000 frs. A description of the harbour works during construction was given in this journal for December, 1929 and January, February, and March, 1930, the last article describing the new station buildings for which, as well as for the jetty, Messrs. Christiani & Nielsen were the contractors.

The station buildings cover an area of about $6\frac{1}{4}$ acres, and are founded on 924 pre-cast reinforced concrete piles 60 to 65 ft. long, driven to rock. The ground through which the piles had to penetrate is a filling consisting of a mixture of fine sand and gravel, below which is an old beach composed of a layer of sand and 18 ft. of clay resting on the rock. Through these strata it was impossible to drive the piles to the rock when a steam hammer was used, and the assistance of a water jet was found to be necessary. In a description of the work published in "Le Génie Civil" it is stated that the pump was driven by a 90-h.p. electric motor and had a discharge of 3,500 cb. ft. per hour. The pressure used was 140 lb. per square inch. With the aid of the water jet and a preliminary excavation 6 ft. deep at the position of the pile, the ground was rapidly removed under the point of the pile, and the latter descended to the clay bed under its own weight in a few minutes. A short period of driving with a hammer—on an average 75 blows—was then sufficient to enable the pile to penetrate the clay and reach the rock. In this way it was possible to drive ten piles in a day, the greater portion of the time being spent in moving the pile frame and in handling and lifting the piles. The operations of sinking and driving the pile were generally executed in about 15 minutes.

Bridge Types and Choice of Type.

By A. W. LEGAT, M.Inst.C.E.

INTRODUCTION—DIFFERENT TYPES—INVESTIGATION—NATURAL FACTORS
— SUBFOUNDATIONS — DISTURBANCE — HEADROOM — LENGTH — ARTIFICIAL
FACTORS—LOAD—WIDTH—CONTOUR—APPEARANCE—TYPICAL EXAMPLES
— SPECIAL CONSIDERATIONS

Bridge Types Available.

EVERY site provides its own particular conditions affecting the choice of the type of bridge most suited to the situation. The number of different cases is so great that it is impossible to do more than sketch the general principles involved and to give some brief indication of the procedure the engineer should adopt in selecting the type of bridge for any particular site. There are roughly thirteen main types of bridge construction in reinforced concrete, most of which are again subject to variation in transverse arrangement in two or more of five different ways. In considering what type of bridge is best suited to any particular site therefore the engineer has a very wide choice before him, and the selection of the type best suited to the site conditions will call for careful consideration of every item of data he may be able to collect.

For most sites it will be found that several of the different types of construction will meet the local conditions, and the selection of these is not difficult. They may in fact be chosen by inspection of the types illustrated later, with little more than consideration on the lines of the accompanying notes. To choose from these the particular type which will meet the conditions with maximum efficiency, permanency, and economy is a much more complicated matter, and it will probably call for rough calculations and rough estimates to assist in the comparison of two or more of the possible schemes before the correct choice can be made. Whilst it is necessary to make investigation for each individual case encountered in practice, some assistance can be obtained by a consideration of the various factors which affect the choice, and it is the object of this article to define these factors and to suggest the general lines of investigation which may be followed.

The following notes relate to the thirteen different types of bridges referred to, and which are illustrated in *Figs. 1 to 13* with the various transverse arrangements "A" to "E" shown in *Figs. 14 to 18*. Against each of the diagrammatic outlines is given the minimum span for which it would be usually considered; for some of the types are stated the maximum span constructed up to the date of writing, so far as the author has been able to ascertain, and the span range over which each type is commonly applied. It may be anticipated that the development of reinforced concrete, especially in regard to the use of higher

working stresses, will in future tend to increase the maximum span for each type.

Type No. 1 (*Fig. 1*) is a freely-supported slab or girder construction resting on abutments of any suitable form or material. This type is generally suited to spans up to say 35 ft. for a slab or a slab and beam construction, and to approximately 70 ft. if some form of parapet girder is used. It is simple to design and construct, and provided the bearings are properly arranged it offers a certain degree of "flexibility" in the event of bedding down or other disturbance of the substructure.

Type No. 2 (*Fig. 2*) consists of a series of continuous spans each of which may have a length up to say 70 ft. if parapet girders are used, but which would probably be limited to about 50 ft. in the case of girders below the slab and to 30 ft. if solid slab construction were adopted. In this type stresses set up by contraction and due to temperature changes must be taken into account. These may necessitate a cross division of the bridge by expansion joints at intervals if the total length is considerable. Care will also be required in the design of the bearings on the piers and abutments to avoid any considerable lateral stresses being transmitted to the piers.

Type No. 3 (*Fig. 3*) is an open frame construction in which the horizontal deck slab is made monolithic with the vertical abutment walls. This construction is suited to spans up to say 50 ft. but would not generally be found economical for spans exceeding about 30 ft. It is a type which might probably be adopted with advantage more frequently than has been done in the past.

Type No. 4 (*Fig. 4*) is a series of continuous spans in which the superstructure is not only continuous in itself but is also monolithic with the supporting abutments and piers. The span limits are similar to those of Type No. 3. This is a form of construction which should not be adopted where there is any doubt of the rigidity of the subfoundation, for it will be apparent that any bedding down of the piers or abutments relative to the other foundations would induce extremely high stresses. For ideal situations, as for example on rock foundations, the method would be economical.

Type No. 5 (*Fig. 5*) consists of continuous girder spans having varying moments of inertia along the span. As shown, this gives to the soffit of each span a shape similar to an arch, but where the girders are designed to project above the deck the structure may resemble a cantilever girder or a suspension bridge in outline. This is a type of construction suited to much longer spans than those previously mentioned, and it might be used with advantage in favourable circumstances up to a span of 150 ft. or even more. The superstructure might be made continuous with the piers and abutments or might be provided with free supports on the top of the piers and abutments. Generally speaking there is an advantage in providing joints, so avoiding somewhat doubtful variations of stress on the substructure and subfoundations.

Type No. 6 (*Fig. 6*) is suitable where the bridge is divided into several spans. Its main feature is the construction of alternate spans with projecting cantilevers the ends of which are used as supports for freely-supported spans constructed between them. This construction has been used for spans up to 200 ft. It has evidently greater "flexibility" than any of the continuous spans already mentioned and is therefore particularly well suited to sites liable to

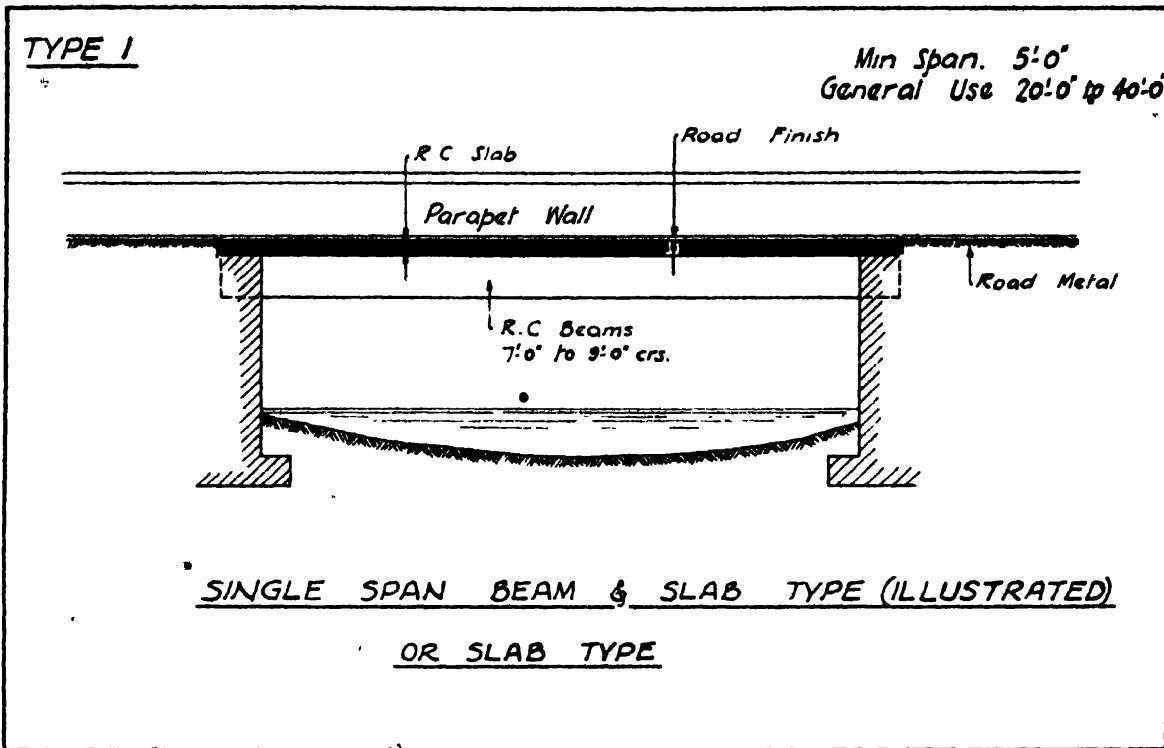


Fig. 1.

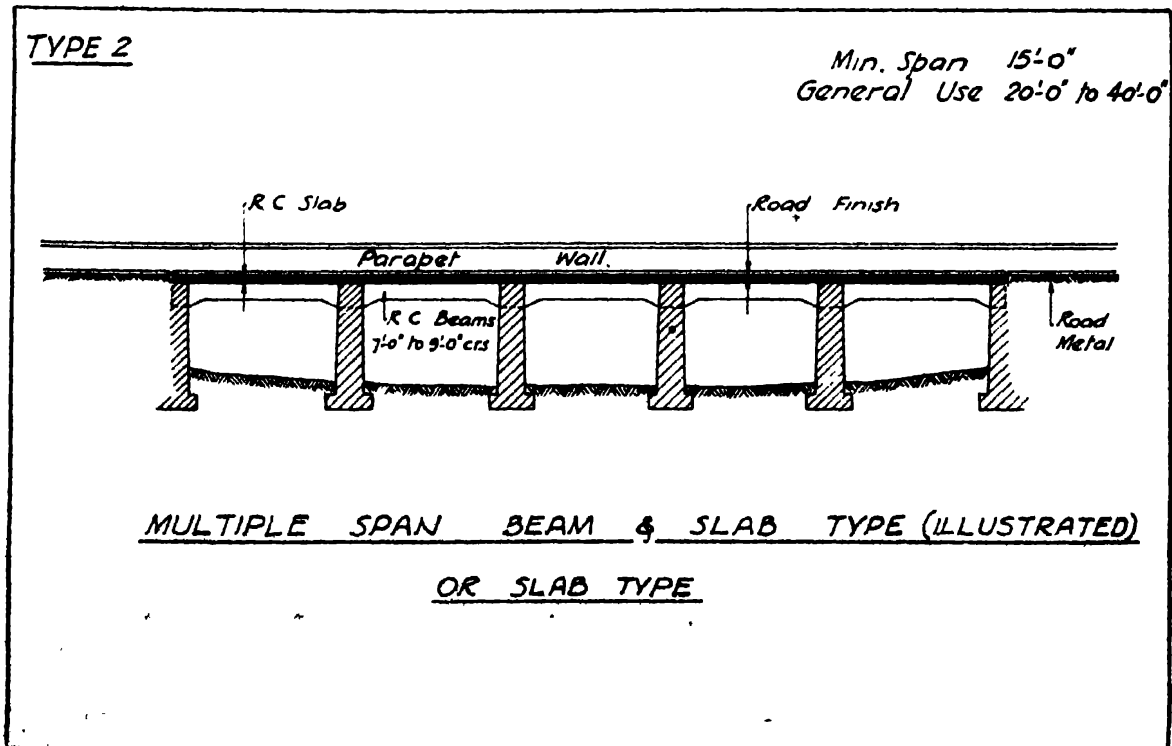


Fig. 2.

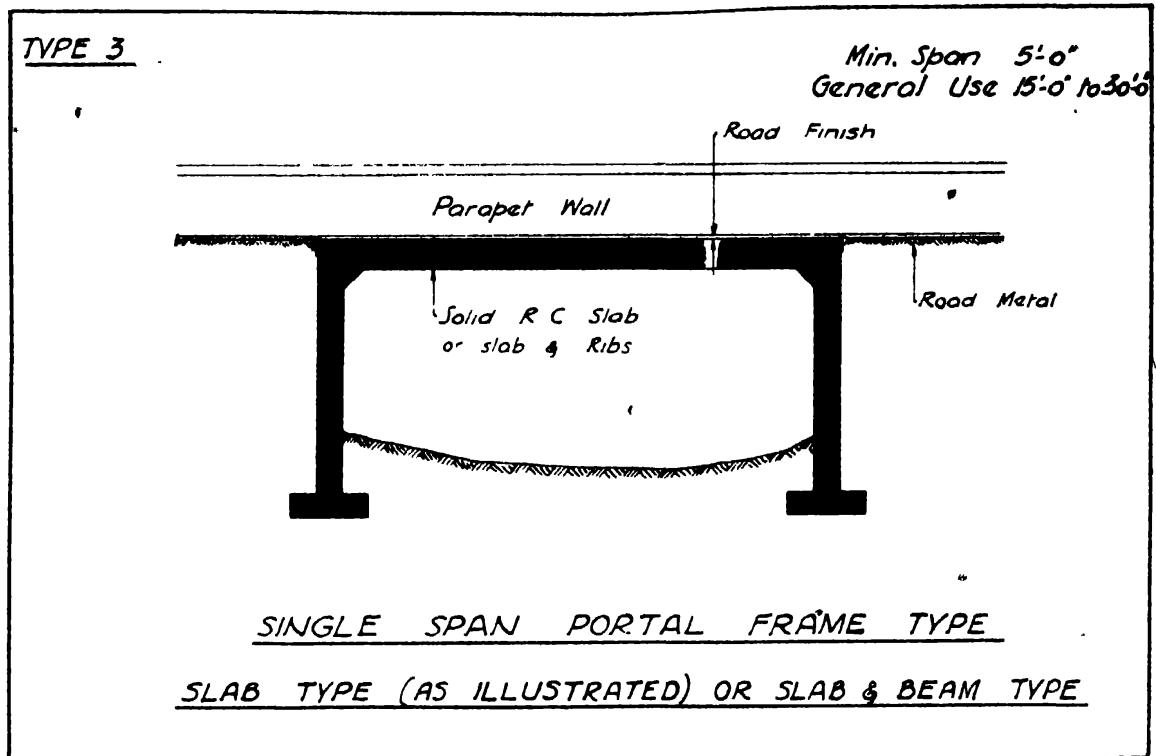


Fig. 3.

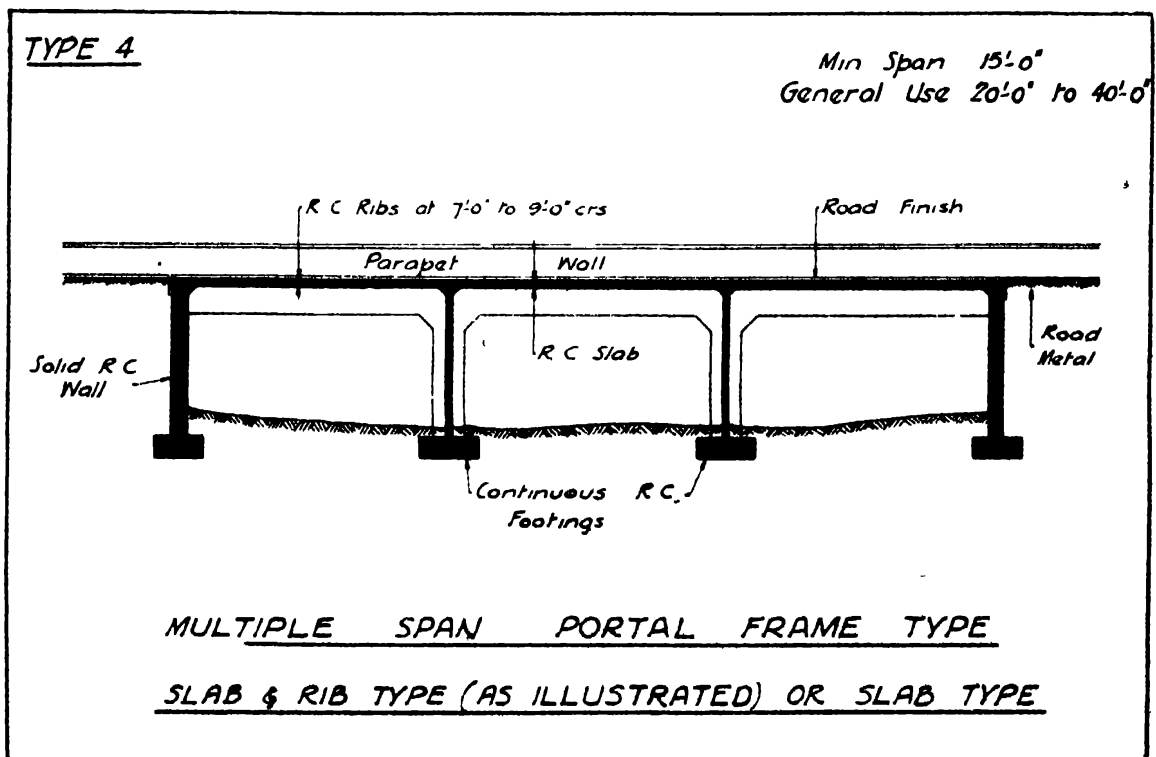
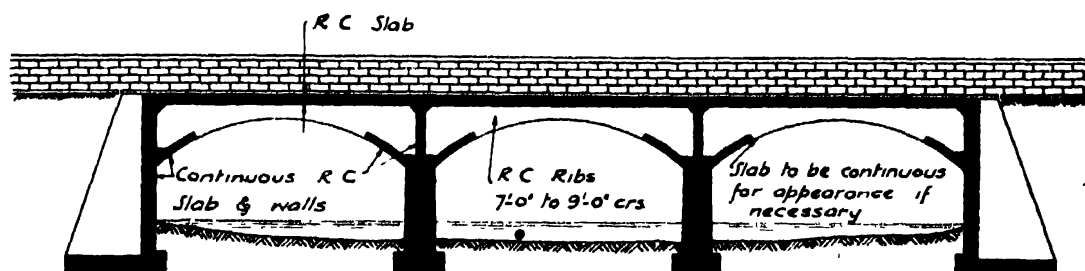


Fig. 4.

TYPE 5

Min. Span 30'-0"
Max. Span to date 170'-0"
General Use 50'-0" to 120'-0"

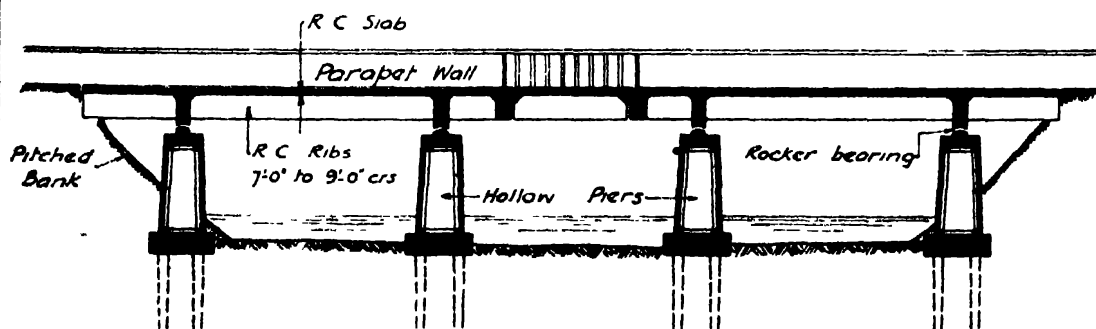


GIRDERS WITH VARYING MOMENT OF INERTIA

Fig. 5.

TYPE 6

Min Span 40'-0"
Max Span to date 200'-0"
General Use 60'-0" to 100'-0"



DOUBLE CANTILEVER TYPE

WITH FREE CENTRE SPAN

Fig. 6.

some bedding down or settlement of the supporting piers. For short spans up to say 40 ft. plain bearings may be used between the piers and the superstructure, but for longer spans a form of rocker or roller bearing which will permit of slight rotation due to loading, bedding down of the foundations, or other causes is an advantage.

Type No. 7 (*Fig. 7*) consists of a single long span between supporting piers, but with projecting cantilevers on the landward side of the piers at both ends forming counter-balances the effect of which is to reduce the positive moment in the centre span. This construction has been used for spans of up to 450 ft. between central supports. It is open to variation by the introduction of joints and a freely-supported span in the centre of the main span. This gives the construction greater "flexibility." It is a type open to considerable development and might with advantage be adopted in more cases than has been done in the past. Where the span is across a canal or river and towpaths are required on both sides the cantilevers projecting behind the main piers can be arranged to span over these paths.

Type No. 8 (*Fig. 8*) is a fixed arch, or arch made monolithic with the abutments or piers, and provided with closed spandrels, i.e. with side walls built up along both sides of the arch, the space between and over the arch barrel being filled with soil or other suitable material up to roadway level. It is suitable for use on spans up to 200 ft., or in special cases even more, but it is not often adopted for spans exceeding 120 to 150 ft. The reason for this is that the use of solid filling greatly increases the dead weight of the structure over and above that required if a construction similar to Type No. 9 is adopted.

Type No. 9 (*Fig. 9*) is a fixed arch in which the arch barrel or ribs are monolithic with the supporting abutments or piers. Instead of the solid filling noted in Type No. 8 the deck is carried by column, beam and slab construction, or cross-wall and slab construction. This type has been used successfully for the longest spans yet constructed in reinforced concrete, the most notable case to date being the Elorn Bridge at Brest having spans of approximately 600 ft. ♦

Type No. 10 (*Fig. 10*) shows a three-hinged arch in reinforced concrete. It has been adopted with success for spans up to 200 ft. and is very suitable for situations where there is a possibility of slight movement in the subfoundation, as may possibly occur in certain types of soil or in areas subject to mining or other disturbance. Another advantage is the reduction of stresses due to contraction and temperature changes which become so important in monolithic structures. The hinges may be made entirely of metal, may be moulded in concrete, or may be formed in the reinforcement of the bridge.

Type No. 11 (*Fig. 11*) shows a two-hinged arch. This is a type which is not usually adopted. It has an advantage over the fixed arch in some cases, but as a general rule it will be found that if hinges are to be introduced then it is better to use three hinges, one at each springing and one at the crown, rather than to limit them to one at each springing as in this type. Here again the use of the hinges appreciably reduces the stresses due to contraction and changes of temperature.

Type No. 12 (*Fig. 12*) is the bowstring form of construction consisting of arch ribs constructed above the deck level of the bridge with horizontal ties

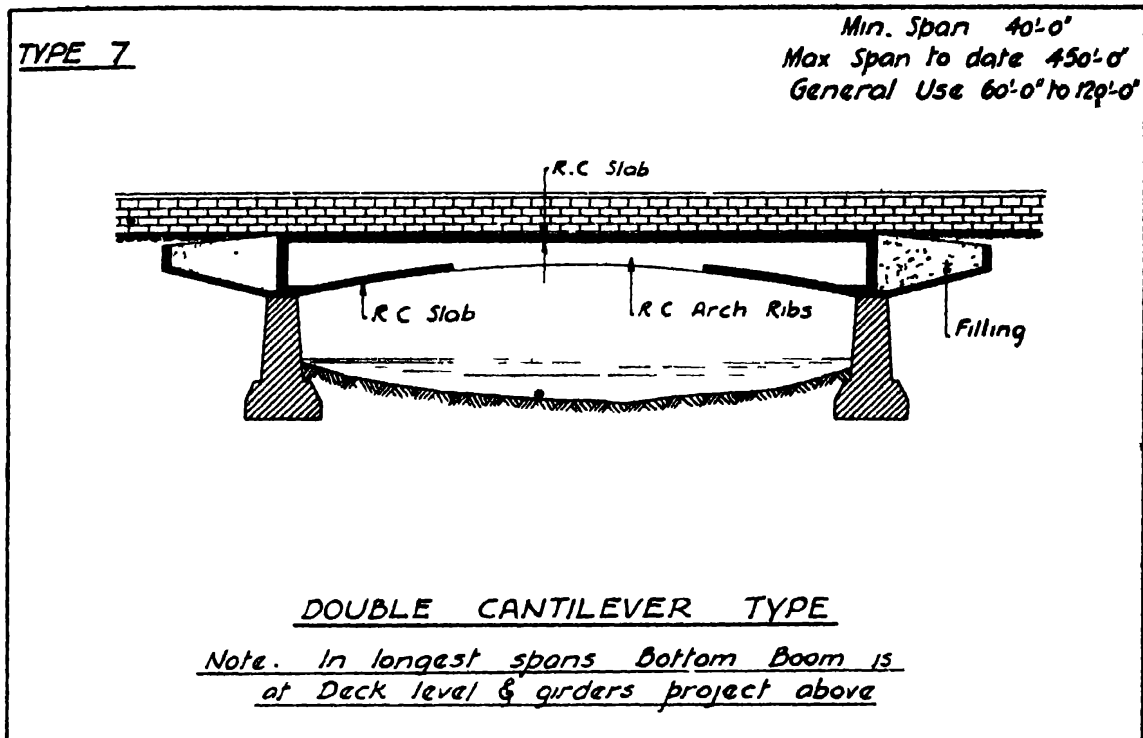


Fig. 7.

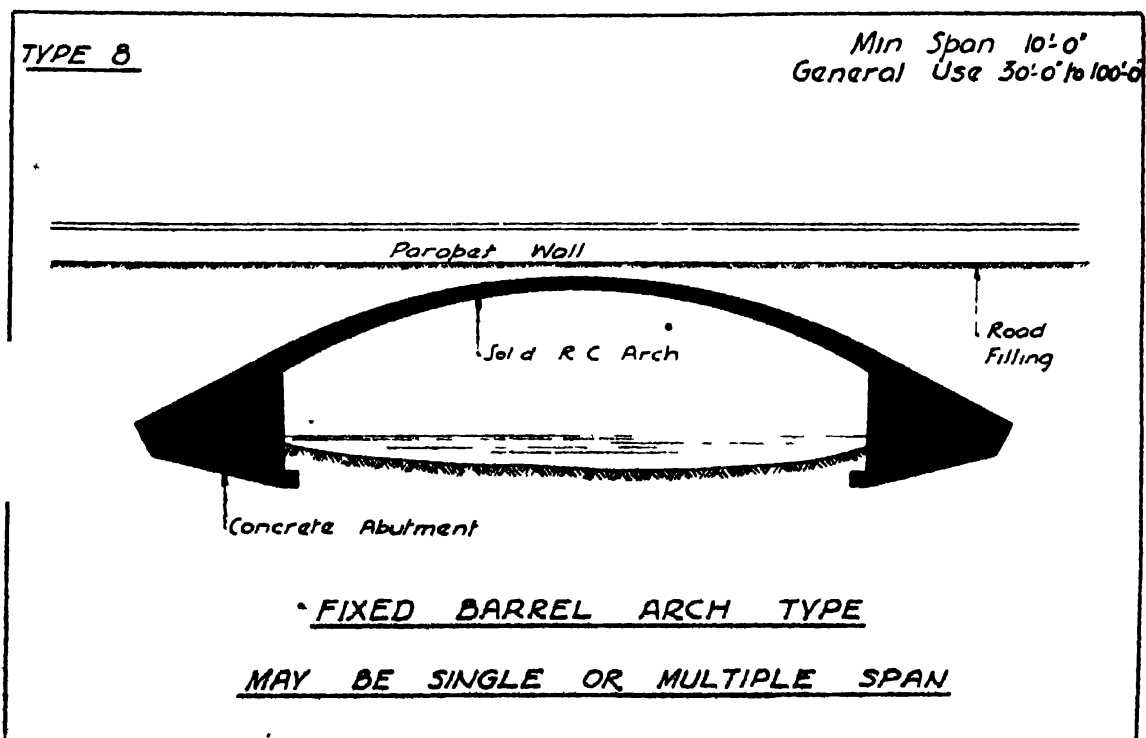


Fig. 8.

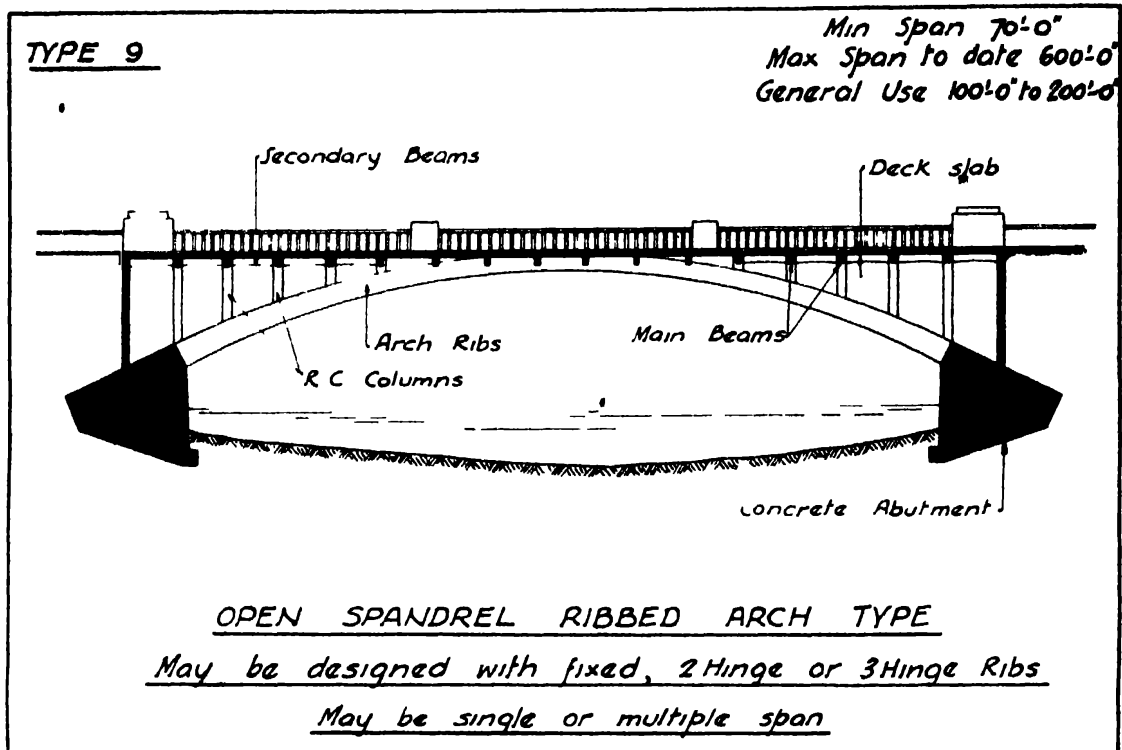


Fig. 9.

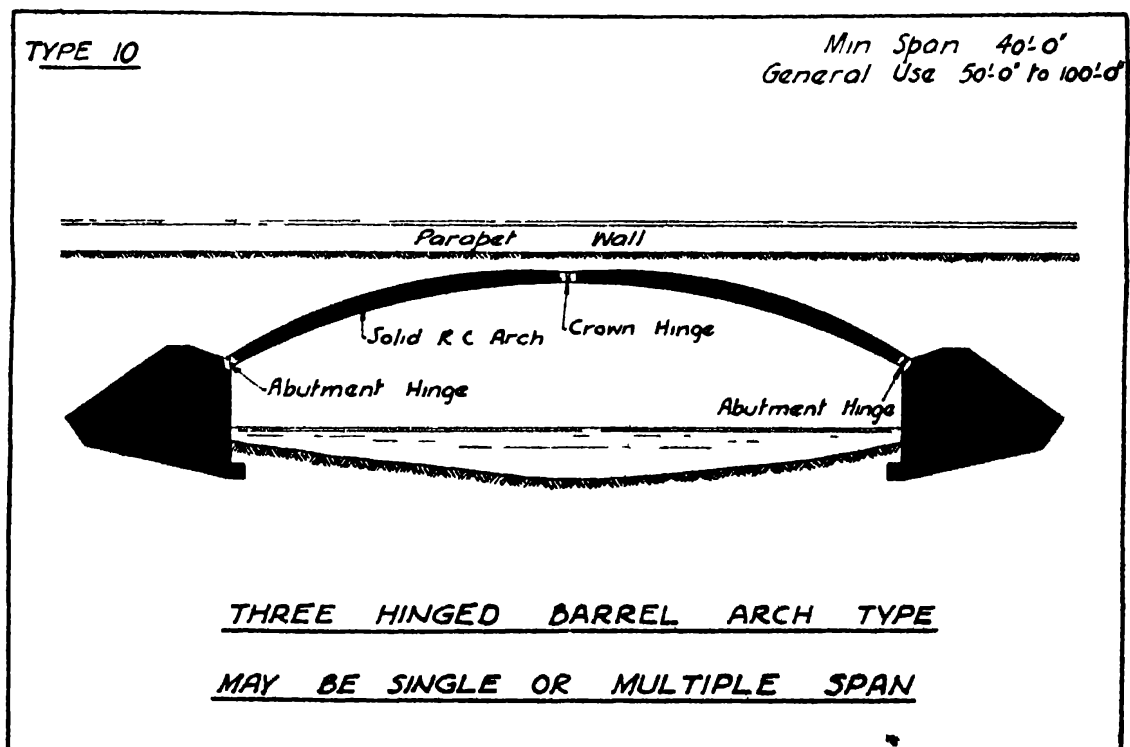
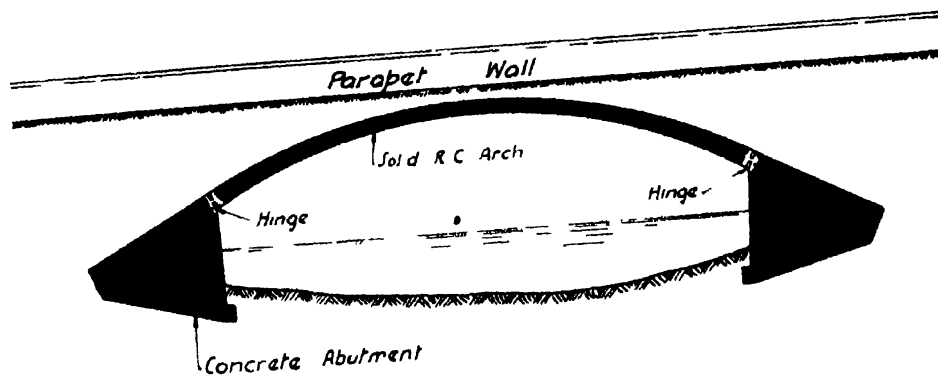


Fig. 10.

BRIDGE TYPES AND CHOICE OF TYPE.

TYPE 11

* Min Span 40'-0"
General Use 50'-0" to 100'-0"

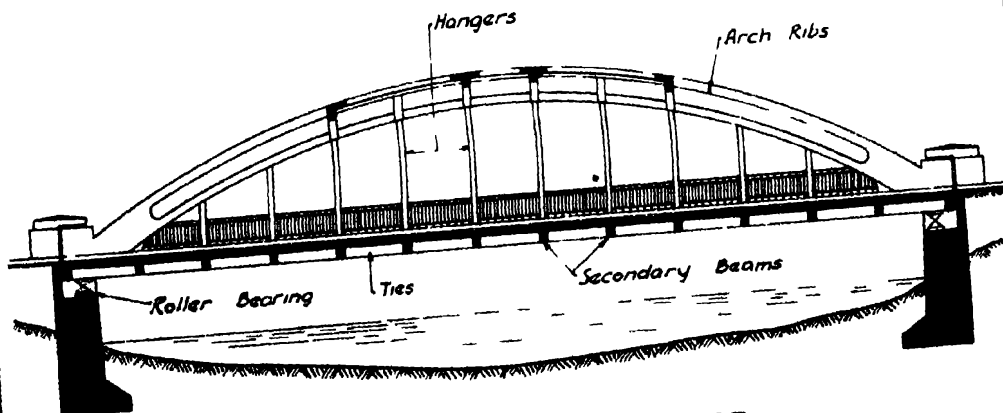


TWO HINGED BARREL ARCH TYPE
MAY BE SINGLE OR MULTIPLE SPAN

Fig. 11.

TYPE 12

Min Span 70'-0"
Max Span to date 300'-0"
General Use 100'-0" to 150'-0"



BOWSTRING GIRDER TYPE
MAY ALSO HAVE HINGE AT CROWN

Fig. 12.

connecting the springings and resisting the horizontal thrust of the arches. These ties are therefore in direct tension. Usually they are suspended from the arch ring at intervals and form the side members of the deck construction. This form can be adapted to long spans, the longest noted to date being 300 ft. By some engineers the direct tension in hangers and horizontal ties is considered to be an objectionable feature tending to the development of hair cracks in the concrete which might result in deterioration of the reinforcement; there is little direct evidence to support this criticism, however.

Type No. 13 (*Fig. 13*) is a variation of the bowstring construction in which the deck is placed at a level above the apparent springing of the arch. The deck is suspended by ties from the arch ring for the major portion of its length, but towards the end is supported from the top of the arch ring by means of columns or cross walls. This is a type which is not often seen and which the site conditions seldom make necessary.

Most of the thirteen types described are subject also to variation in the arrangement of the cross section of the bridge. These are indicated in *Figs. 14 to 18* and are marked Types A, B, C, D, and E.

Type A (*Fig. 14*) consists simply of a slab of uniform thickness throughout the cross section. It is found in short-span portal-frame construction and in some parapet-girder bridges.

Type B (*Fig. 15*) consists of a deck slab supported by ribs or beams on the underside, and is possibly the most common form of deck slab construction in bridge work.

Type C (*Fig. 16*) consists of a deck slab supported by ribs or girders projecting above it along both sides. Its usual form is that of the parapet bridge.

Type D (*Fig. 17*) is similar to Type C except that the upstanding ribs are not so deep and do not form part of the parapet. It is not a very common form of cross section.

Type E (*Fig. 18*) is a cross section of a cellular type of bridge in which a soffit slab is provided along the bottom of the girders and the deck slab spans across the girders at the road level. Externally a bridge of this type has a solid appearance. Due to the reduction of dead weight which results it is a form suited to long spans, and will probably be used more in future than it has been in the past.

Types A, B, D and E can be used for any width of bridge, whilst Type C is suitable only for comparatively narrow bridges usually of the parapet girder type.

In addition to these general types and variations in reinforced concrete bridge construction one occasionally encounters what may appear to be unusual types, often simulating structural steel work in general arrangement and appearance. Actually these may almost invariably be included with one or other of the types illustrated. The great amount of intricate shuttering work which they involve makes them unsuitable for what are to-day considered normal spans. Due, however, to the saving of dead weight resulting from their open-work construction they appear to be more suited to very long span bridges, and it is mainly in this class of work that they have been used up to the present. For spans under 250 to 300 ft. it is doubtful whether they would prove economical.

Having before us the different types of bridge which may be adopted, there

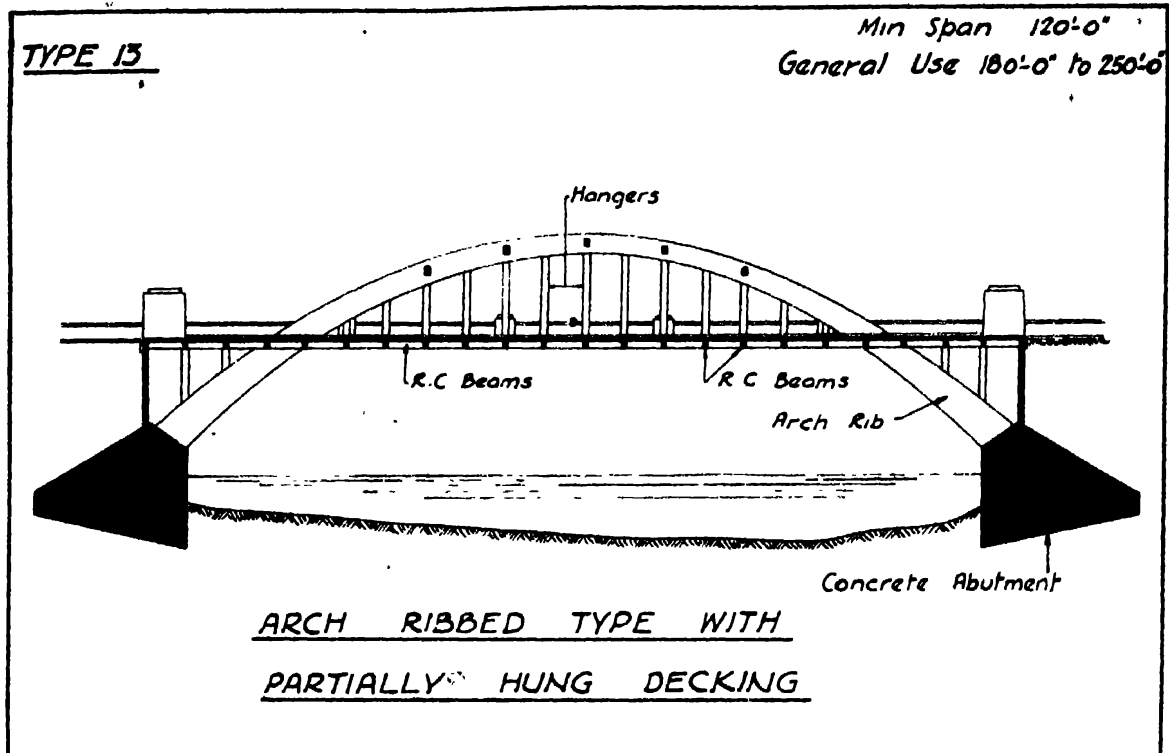


Fig. 13.

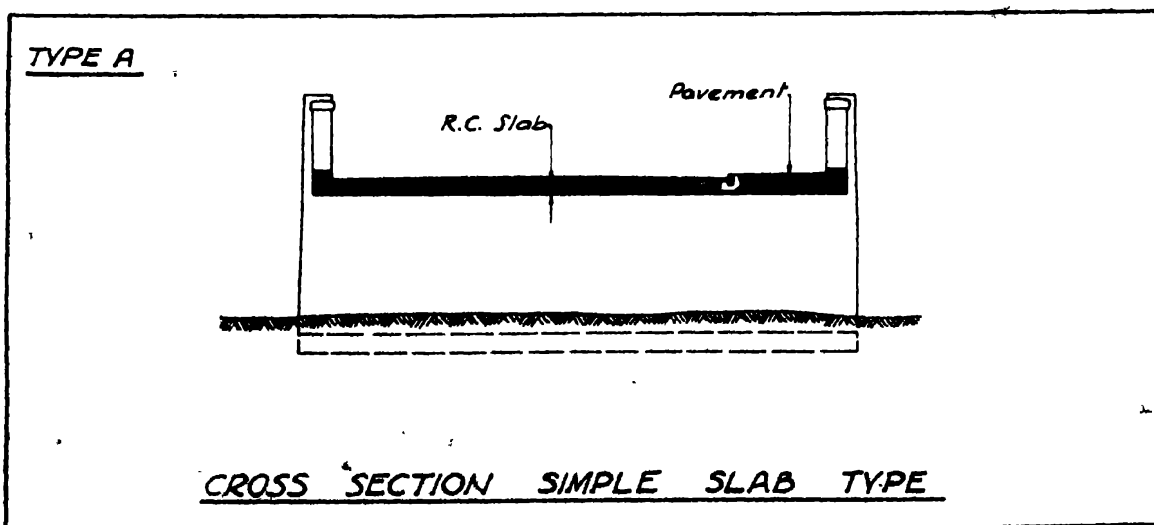


Fig. 14.

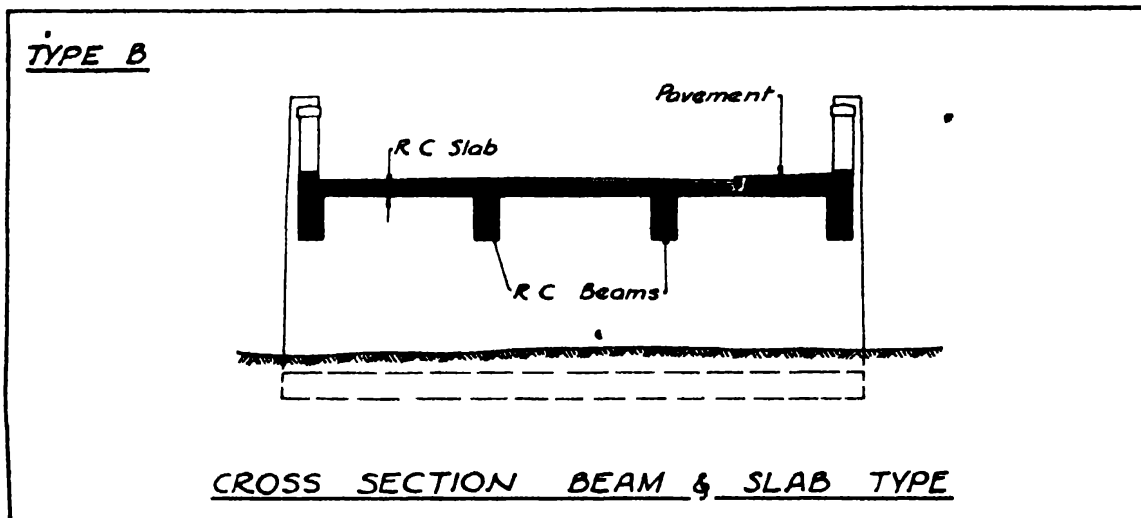


Fig. 15.

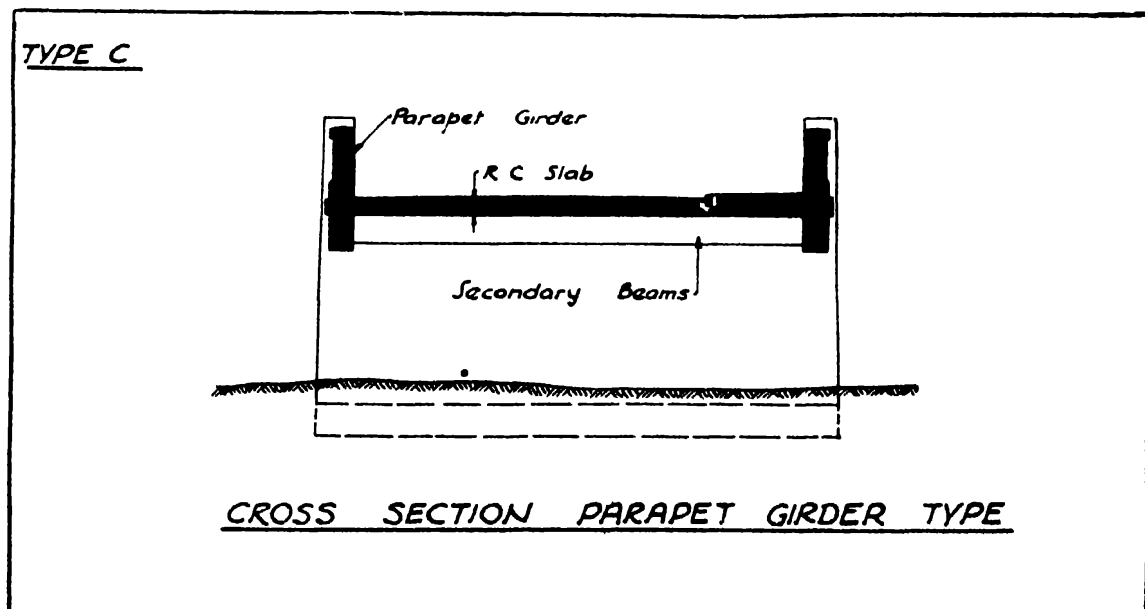


Fig. 16.

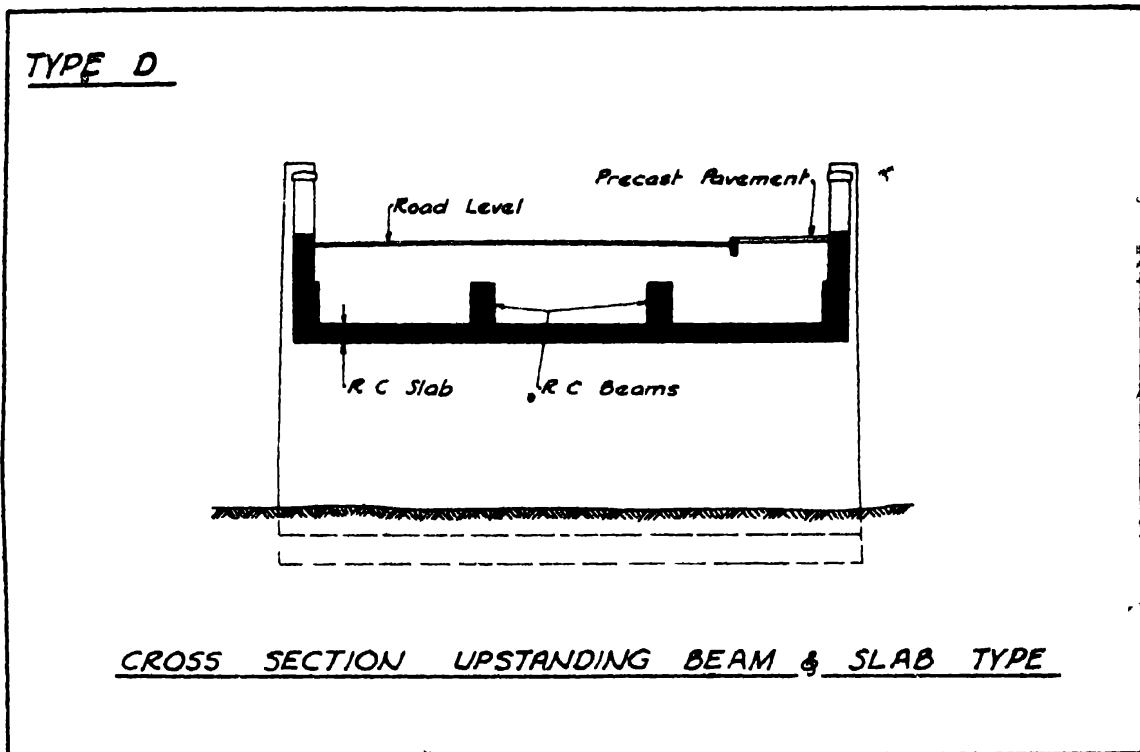


Fig. 17.

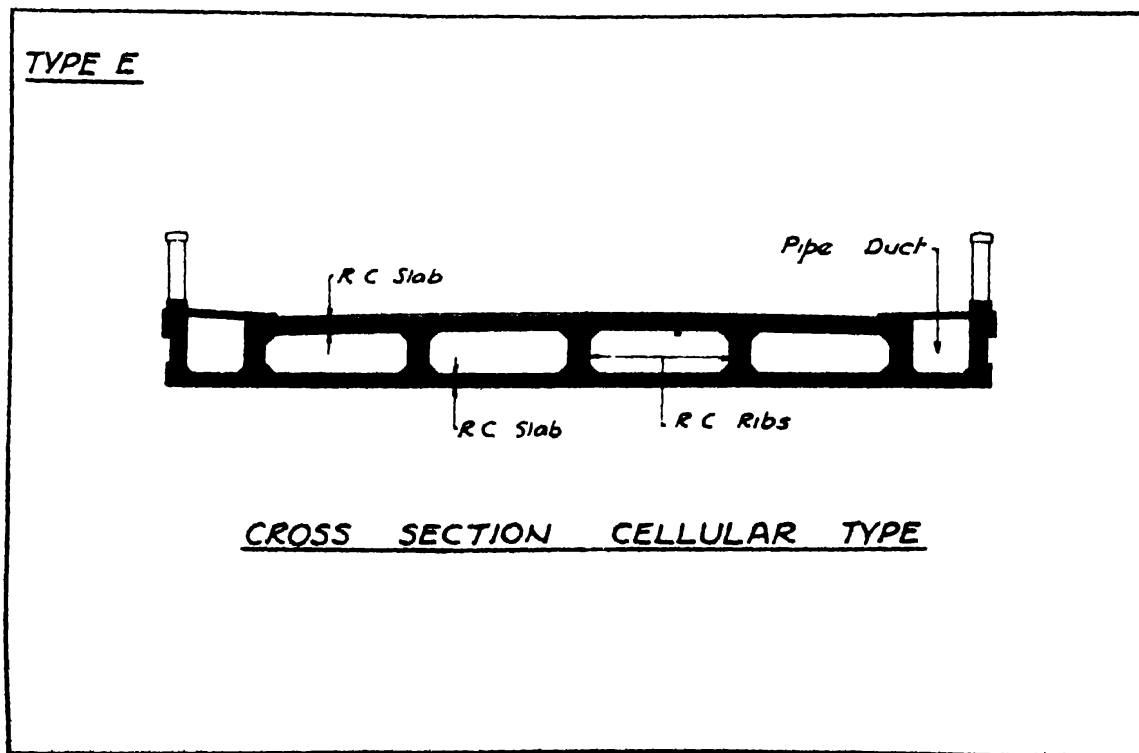


Fig. 18.

are three questions to which answers must be found before one can decide on the type of bridge most suited to any particular case.

- (1) Which of the types can be adapted to the particular site in question?
- (2) Which of these possible types will do the work most economically?
- (3) Are there any disadvantages in the adoption of this most economical scheme? If so, one must then consider one of the schemes which is a little more costly than the cheapest to meet the requirements.

If we can find definite answers to these questions for any particular example then we shall be in a position to concentrate on the detail design of the chosen type knowing that the final design will have all the requirements of the ideal bridge and particularly those of efficiency, permanency, and economy.

In endeavouring to find an answer to these three questions, there are two sets of factors which call for consideration. The first, which may be termed the natural factors or factors dependent entirely on the site conditions, are:

- (a) The subfoundation.
- (b) The possibility of mining or other disturbance.
- (c) Headroom necessary
- (d) The length of the bridge.

There are also a number of what may be termed artificial factors, that is, factors decided by the necessities of the case. These may be enumerated as:

- (a) The load which the bridge is to carry.
- (b) The width of the bridge.
- (c) The contour
- (d) The appearance.

Let us now consider these factors separately with a view to appreciating the extent and manner in which they affect the choice of bridge type.

Subfoundation.

It may be taken as a general rule that if the foundation work is comparatively costly it will be better to reduce the number of foundations as much as possible and in consequence to use comparatively long spans. If, on the other hand, the foundations are likely to be inexpensive it will be more economical to use short spans and a comparatively large number of foundations.

It has often been suggested that in multiple-span bridges the division of spans should be such that the total cost of the substructure equals the cost of the superstructure. Whilst there is some justification for this it should not be followed too rigidly in view of the many factors involved in the choice of type of bridge. These statements are, however, open to some qualification. Take for example the case where the ground is particularly poor and where in consequence it is necessary to spread the load from the new bridge over the whole area of ground covered by the bridge. This might conceivably occur on a site where it is impossible to obtain an absolutely rigid foundation at any reasonable depth which could be reached by piling, but where a comparatively low loading of the ground at shallow levels is permissible. Although the foundations in such a case would be comparatively expensive, it would obviously be an advantage to have several short spans rather than one long one in order to reduce the stresses induced in the raft system connecting the various piers and abutments.

From the general rule one is also led to the fact that, in most cases, if the foundations require to be carried to comparatively deep levels they will be costly and in consequence it will be better to use comparatively long spans and so reduce the number of foundations. At the same time it should be noted that if a foundation is only obtainable at a very low level it will be better to have only vertical reactions, and in consequence for single spans the arch type of construction is not likely to be favoured in cases of this kind. For multiple-span bridges, however, where the major portion of the horizontal thrust would be balanced over intermediate piers the objection would not apply so strongly. On the other hand, if the foundations are shallow they are likely to be comparatively cheap and short spans will be favoured. Such shallow foundations will also be more suited to deal with inclined reactions, and an arch type of construction would therefore offer some advantage. Where abutments or intermediate piers are to be constructed in a river or other water, dams are likely to be required. These are temporary works carried out solely for the purpose of facilitating the construction of the permanent structure and are removed after the permanent structure is completed. It is therefore desirable to limit the expenditure on such temporary works as much as possible, and this can best be done by reducing the number of foundations required. This automatically guides one to a reduction in the number of spans, so that in such a case, other factors being equal comparatively long spans are favoured.

The reason for the total cost increasing considerably with the increase in the number of dams is apparent, for as a general rule the longer side of the dam will be across the bridge and the short side in the direction of the span of the bridge. For example, suppose a bridge pier is 50 ft. long, say a little more than the total width of the bridge and has a width of say 12 ft., if two such piers are required the total length of dam required will be 248 ft. If, however, four piers are used the width of pier to give the same intensity of loading on the ground will be in the neighbourhood of 6 ft. and the total length of dam required for four piers would be 448 ft., or almost twice as much as in the first case. In addition to the fact that the length of the dam increases almost directly as the number of supporting piers and abutments, the delay occasioned in constructing a number of such dams, timbering inside, pumping out, withdrawal of the dams after use, etc., is likely to be considerable and to increase appreciably the cost of the finished work.

Table I gives the approximate weight per square foot which the superstructures of bridges of various spans are likely to produce, and if to these are added the design load per square foot calculated as equivalent to the live load, one can rapidly find approximately the total load on the foundations for any given span. From the information obtained during the investigation at the site the reasonable intensity of loading to be imposed on the ground at different levels will be known, and by equating the two one will be able to find roughly the width of foundation required for any subdivision of length. If the total width of the foundation for the chosen span does not exceed 20 per cent. to 25 per cent. of the span then it will usually be reasonable to spread the load from the pier. If, however, the spread required exceeds 25 per cent. of the span it is probable that it may be economical to go deeper with the foundations to a stiffer stratum or to use piles, or to deal with them in an even more elaborate

TABLE I.

APPROXIMATE WEIGHTS PER SQUARE FOOT OF GIRDER OR OPEN SPANDREL ARCH BRIDGES.

Span in ft	Approx dead load of bridge superstructure per square foot lb	Ministry of Transport approximate equivalent live load per square foot lb.
20	160	490
30	180	400
50	215	320
70	250	290
100	310	260
150	400	230
200	490	210

[Note : Add the dead and live load figures given to the approximate weight of the foundations per square foot of bridge. This total multiplied by half the bridge area equals the load on one foundation.]

EXAMPLE.

The use of the above table may best be shown by an example.

Suppose one is considering the adoption of a 100-ft. span girder bridge. The approximate width of foundations required may be determined as follows :

	lb.
Dead load	= 310
Live „	= 260
Foundations, say $\frac{2}{3}$ weight of superstructure	= 200
	770

Safe intensity of pressure from foundations = 2 tons per square foot

$$\therefore \text{width of one foundation} = \frac{50 \text{ ft.} \times 770}{2 \times 2240} = 8.6 \text{ square feet.}$$

In the case of an arch bridge, the values of the live load and the weight of the superstructure could be taken from this Table, and the approximate horizontal thrust obtained from the formulae $H = \frac{Wl^2}{8r}$, where r is the rise of the arch. The size and shape of the foundations are then best determined graphically, being largely dependent on the height of the arch springing from the natural subfoundation level.

way. Having made rough calculations on this basis, the engineer will find it necessary to prepare rough comparative costs of foundations necessitated by different spans or types to guide him to an answer to the question "What type and span of bridge is to be adopted?"

Subfoundations Liable to Disturbance.

In mining areas or in areas where brine is pumped from the ground there is considerable possibility of settlement taking place in the ground at great depths below the bridge foundations. It should first be appreciated that such movements are really miniature earthquakes, and it is quite impossible to prevent this type of disturbance from affecting the bridge structure no matter of what material or to what design it may be built. No amount of spread or increase in the depth of the foundations (short of carrying the foundations down to a depth below the mine workings) will prevent ultimate settlement taking place where the whole of the ground sinks for great depths. In consequence it is necessary for the engineer to keep in mind that the bridge may settle and that it may not settle uniformly. In such cases he should adopt a type which will allow for considerable relative variations in settlement, of say 6 in. upwards, of the abutments and various piers, dependent on the knowledge of the settlement already experienced in the neighbourhood, and the design must be such that this settlement may take place without affecting the strength or permanency of the structure. A factor of this kind will have a considerable effect on the choice of type of bridge to be built, and will always lead away from monolithic construction towards one of the "flexible" types having the maximum number of hinged joints.

In this connection it should be noted also that in such cases there is a slight possibility of greater settlement at one end of an individual pier or abutment than at the other. In other words, it does not necessarily follow that a pier will settle uniformly over its whole area; it may in fact tilt across the bridge. The only way of providing for this contingency is to design and construct the pier as stiffly as possible along its length so that it will be capable of standing stresses induced by such unequal settlement, and by increasing the intensity of load on the part of the subfoundation which does not settle will tend to crush this and to keep itself as nearly as possible on an even keel. Reinforced concrete is particularly well suited to the construction in such cases. In steel bridges arrangements are sometimes made for the jacking up of the girders on the top of the abutments and piers in the event of non-uniform settlement. There is no reason why a similar arrangement should not be adopted in reinforced concrete construction. The dead weight is generally greater, but usually it would not be found either difficult or unduly expensive to provide facilities for jacking up the reinforced concrete superstructure. Unfortunately there has been up to the present a tendency to decry the use of reinforced concrete for cases of this kind, but it is anticipated that in future there will be a slowly increasing adoption of reinforced concrete for bridges of this type. In such cases Types Nos. 6, 10, and 12 are suitable for consideration; the remaining types should not be adopted.

Another very important factor in deciding the type of bridge to be used is the maximum construction depth available. The level of the soffit of a bridge

is usually governed either by the height of flood water or by any navigational use to which the river or stream may be placed. On the upper side the level will be governed by the contour of the roadway over the bridge, and the difference between the two will give the construction depth available. If this construction depth is large then it will in itself have no effect on the type of bridge to be used, but if it is restricted and it becomes less than a twentieth part of the span then the girder type of construction (Nos. 1, 2, 3, and 4, and possibly also No. 6) is not likely to be suited to the case. Instead it will be found that Type No. 12, the bowstring girder, will give the maximum of construction depth owing to the fact that the actual carrying members of the bridge project well above the roadway. Types Nos. 5, 7, 11, and 13 will probably be the next most suited for a case of this kind. As regards the cross section of the bridge, Type A would probably give the minimum of constructional depth, followed closely by Types C and D. The construction depths necessary for a roadway say 20 ft. in width, using Type No. 12 bowstring girder, will probably be from 18 in. to 24 in. and will be quite independent of the bridge span and wholly dependent on the width between the bowstrings and also on the load to be carried. The arch construction (Types Nos. 8, 9, and 11) and the girders having varying moments of inertia (Type No. 5) will probably necessitate a constructional depth in the centre from a thirtieth or a fortieth part of the total span of the bridge, depending to some degree on the constructional depth available at the springing or alternatively on the rise of the arch.

Length of Bridge.

The total length of the bridge is another factor affecting the type to be adopted, and the length itself depends almost entirely on the economic point at which the change should be made from suspended to solid construction. It resolves itself into a comparison of costs of suspended approaches and of solid approaches consisting of approach walls and the necessary filling. A table similar to *Table II* based on rates applicable to the site will be of assistance. This gives the approximate cost of retaining walls of various heights on normal ground together with the cost of filling per foot of length for varying heights and for approaches having a total width of 20, 30 and 40 ft. The cost of the filling is based on a rate of 3s. 6d. per cubic yard in place. If we take the cost of construction of suspended approaches at say £10 per square yard and compare this with the cost of the solid approach construction we can quickly find a suitable point at which to change from solid to suspended work. As a general rule this will be found to occur where the bank in the approach increases beyond 20 ft. in height. It should be noted that whilst the prices given are suitable for normal conditions and rates at the present day they will vary for different districts as the prices of the various materials involved vary from time to time. In consequence it will be necessary for the engineer when he has an important bridge structure under consideration to make up approximate prices based on the local rates ruling at the time on the same lines as those shown for purposes of comparison.

There are numerous factors which may considerably influence this decision. For example, in some cases the materials excavated may be gravel suitable for

use in concrete, and even if not suitable for reinforced concrete it will be reasonably suitable for mass concrete. In a case of this kind the retaining walls to the approaches would be more economical to construct than in normal cases, and in consequence it would probably be an advantage to carry out the solid approach work farther and higher than in a normal case.

A similar effect would result if filling were particularly cheap or if it was necessary to find a suitable tip for the excavations for the bridge foundations. Here it would be advisable to arrange the approach works so that they would absorb the whole of the surplus excavation available.

Another case might occur where it was wished to make some improvement to a corner of the roadway at a site not far distant from that of the bridge and where there might not be available a suitable tip for any surplus soil. In such a case use might be made of the approaches of the new bridge to deposit the soil from such an improvement.

The type of subfoundation will also have to be taken into account. In cases where suspended work necessitates deep and costly foundations, while solid construction can be founded cheaply at shallow levels, then the latter will obviously offer an advantage and will in consequence be adopted as far as possible.

It will also be noted from consideration of *Table II* that the narrower the bridge the longer will be the total span, from solely economical reasons due to the fact that the approach work will always call for two supporting walls which are much more expensive than the intervening filling and more closely approach the unit cost of suspended work. Usually, therefore, the narrower the bridge the longer the suspended work and the shorter the solid approach; the wider the bridge, the shorter the suspended work and the longer the solid approach work.

A further factor which might have some effect on the position at which one changes from solid to suspended work is the cost of the land on which the bridge is to be built. If this was particularly high it would be desirable to limit the width taken up by the banking of the approaches as much as possible and in consequence this banking would probably be retained between walls for the whole of its length.

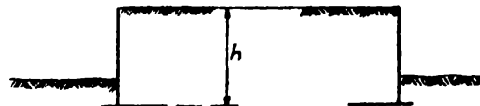
These various factors affecting the point where the change should be made from solid to suspended work have been briefly mentioned, but their number might be considerably extended; in fact the data collected at the site of any proposed bridge will almost invariably contain some novel factor affecting this choice. Those mentioned will, however, provide some guidance to the line of thought necessary when considering this matter.

In addition to consideration of the points at which the change from suspended to solid work should be made, one must also consider the question, "When should one change from main spans to suspended approach spans?" This is a factor almost invariably settled by the contour of the site, but in principle it should be noted that down to comparatively short spans the greater the number of supports the lower the cost of the suspended work per unit of area on normal foundations.

Another factor which will enter into the question is that of the width of the river to be spanned, and as has already been noted it is essential that the

TABLE II.

APPROXIMATE COSTS OF SOLID CONSTRUCTION FOR BRIDGE APPROACHES.



Height in ft.	Cost per lin. ft.		Cost per lin. ft. of filling at 3s. 6d. per cub. yd.		
	1 wall	2 walls	Bridge 20 ft. wide	Bridge 30 ft. wide	Bridge 40 ft. wide
8	22s.	44s.	62s.	93s.	124s.
10	32s. 6d.	65s.	78s.	117s.	156s.
12	45s. 6d.	91s.	93s. 6d.	140s.	187s.
15	70s. 6d.	141s.	117s.	175s. 6d.	234s.
18	101s.	202s.	140s.	210s.	280s.
20	127s.	254s.	156s.	234s.	312s.
25	203s.	406s.	194s. 6d.	292s.	389s.
30	304s.	608s.	233s. 6d.	350s.	467s.

Note.

Add cost of 2 walls and filling between, and where this is less per square yard than the cost of suspended work, or say approximately £10, then solid construction is probably preferable.

EXAMPLE.

A bridge approach is to be 40 ft. wide and 25 ft. above foundation level. Will solid or suspended construction be more economical?

From the Table:

Cost of two walls = £30 8s. 0d.

„ filling = 23 7s. 0d.

£53 15s. 0d.

Cost of solid construction per square yard = $\frac{£53\ 15s.\ 0d. \times 9}{40}$

= £12 approx.

Suspended construction (cost £10 per square yard approximately) would probably be the cheaper.

full area of waterway existing before a bridge is built shall be preserved in the design of the new structure.

Loads, Width and Contour.

Next we come to artificial factors affecting the choice of type. In the first place we have the question of load. Where this is heavy it will tend towards the adoption of comparatively short spans, whilst if the design load is light it will tend towards the adoption of longer spans. The reason for this is that the heavier the load the heavier the construction required to carry these loads and the more expensive in consequence does the suspended work become.

The next question is that of width of bridge. This has not a very great influence on the type of construction to be chosen except in cases where the bridge is so small that parapet girders may be adopted. These will provide an economical form of construction, but they are not generally liked owing to the possibility of vehicles colliding with them, and in damaging the parapet at the same time damaging the supporting power of the structure. This is a factor which does not occur if the parapet is not part of the supporting structure of the bridge as is the case in every other type.

The next factor is the contour of the bridge, and this can be adapted to any type except that of the bowstring, which as a general rule is made horizontal or with only a very slight camber along the length of the bridge rising towards the centre.

Appearance.

Finally there is the question of appearance. Where the bridge is to show a concrete finish any of the types may be adopted, but where it is wished to add a veneer of stone or other material then only solid spandrel construction is suitable. This can be obtained in Types Nos. 1, 2, 3, 4, 5, 6, 7, 8, 10 and 11.

A decision on the subdivision of the total length of a bridge is also frequently affected by the appearance. Unless there is special reason to the contrary, multiple-span bridges should have an odd number of spans; an even number never looks well unless there is a special reason obvious to the layman when the bridge has been constructed.

In certain situations some of the types may appear incongruous and unsuitable, and this is a factor which should be given careful thought for it is unwise for the engineer when designing a new structure to offend the æsthetic taste of the general public. He must remember, however, that the purpose of a bridge is of greater importance than its appearance, and it is seldom that the type of bridge to be adopted will be governed by its elevation. Instead, when an architect is not employed, it will be a problem for the engineer to make the elevation of the most suitable type of bridge fit in as well as possible with its surroundings. We know of a case where a bowstring girder has been erected in a wild moorland situation, and which in consequence has been severely criticised on the score of its unsuitable elevation. In this case the bridge crosses a stream traversing a wide valley, the stream being liable to sudden and high floods. To have raised the roadway in the neighbourhood of the bridge in order to permit of an arch or girder type of construction would not only have been expensive but might also have been dangerous due to the possibility of floods overflowing the

river bank and causing damage to the new embankment on both sides. In consequence a bowstring girder was adopted requiring the minimum constructional depth and so providing the maximum headroom for the flow of water. This is a perfectly sound engineering decision. It is admitted that the appearance of the bridge is not quite suited to the countryside. Possibly a roughly bush-hammered stone, concrete, or masonry-faced arch would have been more fitting, but this is a case in point where although the bowstring is unsuitable from the æsthetic point of view it is nevertheless the only sound engineering solution of the problem.

Examples.

Having suggested in brief outline the various questions which arise in selecting a type of bridge, it is proposed to apply the conclusions to a number of typical cases where bridge construction might be required. It must be understood that every bridge type requires separate consideration of all the factors mentioned before a final decision can be arrived at, and the three examples given are merely a guide to the method of analysis and are not the analysis itself for any particular actual example.

EXAMPLE 1.—A deep and narrow rocky ravine, with rock outcropping at both sides and everywhere found at shallow depths below the surface soil.

This is obviously a case where it would be expensive to bring a foundation up from the bottom of the ravine to a comparatively great height and consequently at considerable cost. The case would therefore lead one toward the adoption of a single-span bridge. If the span were up to approximately 30 ft., Type No. 1 might be adopted. The rocky side of the ravine could, however, provide excellent resistance to the thrust of an arch, and in consequence Type No. 8 would also call for consideration due to the fact that there would be no likelihood of settlement in the rocky sides of the ravine. The fixed type arch No. 8 would here be preferable to the hinged type Nos. 10 or 11. As there are no practical limits to the construction depths this factor does not affect the decision. The choice would probably rest between Types Nos. 1 and 8, and a comparison of cost and appearance would finally determine which of these types would be used.

EXAMPLE 2. - A rocky valley, comparatively shallow and narrow, with the rock to be found everywhere at comparatively shallow depths below the surface. It is probable that in such a case the stream in the valley would be liable to considerable flooding.

In this case the foundations would evidently be inexpensive, and in consequence it would be reasonable to use comparatively short spans. There is no possibility of mining or other disturbance, and in consequence there is no necessity to make the bridge particularly "flexible." Owing to the probability of considerable flooding it would be desirable to provide the maximum waterway and also to give plenty of length to the bridge in order not to affect either the flow of the river in flood or the distribution of flood water when the river overflows its banks. In consequence, if the span does not exceed 30 ft. Type No. 1A (Figs. 1 and 14) would require consideration. If the span is greater, Types Nos. 1B or 1C could be adopted for spans up to say 70 ft., although in view of the comparatively inexpensive foundations it would probably be found more economical

to subdivide the total length into two or more spans. If the total width were 70 ft. or more, and if the construction depth available was very little indeed, and if a single span offered some decided advantage, then a bowstring type of construction might call for consideration. The possibilities here would probably be narrowed considerably by the detail data for any particular case, and as in Example 1 would probably resolve itself into the comparison of cost and appearance of, say, two of the types mentioned.

EXAMPLE 3.—A wide deep valley with a stream to be crossed by the bridge, the ground being of normal type.

In this case the foundation, and substructure would be likely to prove expensive due to their considerable height, and in consequence it would be desirable to limit their number as much as possible. Further, due to their height, as previously noted it would be better to produce only vertical reactions. This rules out arch construction unless the arches are made with very considerable rise and spring from a level close to the ground. No mention is made of mining or other disturbance, and in consequence this factor will not affect the choice. Due to the depth of the valley it is obvious that the question of construction depth and headroom under the bridge would not affect the decision. The total length of the bridge would in this case probably require careful consideration, as also the subdivision of this total length into suitable spans. It is probable that Types Nos. 5 or 6 would prove most suitable for this case, probably adopting cellular construction for the piers.

In these considerations no mention has been made of skew spans, and it should be noted that these have some bearing on the decision arrived at in regard to the type of bridge to be adopted as they invariably create complications in the design although little if any in the actual construction. Types Nos. 1 and 2 are more suitable than Types Nos. 3 and 4 where spans of any considerable skew are under consideration. The reason is that in a skew span there is always a doubt as to the direction in which the load will span, and there is a probability that it will tend to carry across the shortest span, that is, normal to the line of the supports as shown on the accompanying *Fig. 19*. In such a case one would be left with a state of affairs not amenable to strict calculation in the corners of the bridge span. In these positions it is possible that fairly high concentration of stresses will occur on which it is impossible to place a calculated value. Type No. 5, with accurate calculation, is quite easily adaptable to skew spans. The same applies to Type No. 6 except that one is here faced with a somewhat peculiar state of affairs in regard to the hinges. These will not usually be normal to the centre line of the bridge, and in consequence their movement, due to various factors of load, contraction, temperature changes, etc., is somewhat complicated. For spans up to say 60 ft. this will not be a serious matter, but for spans of over say 100 ft. it would require careful consideration. It should be remembered that in cases of this kind, and in fact in every skew bridge, the reinforcement of the acute angle both of the support and of the suspended work should receive special consideration, and special reinforcement should be supplied to meet the stresses which it is evident are likely to develop and on which it is impossible to place any accurate calculation.

It will be appreciated that the choice of bridge type depends to some degree on experience, but even more on a careful study of the site conditions. At

first it will prove somewhat laborious to go into the various considerations and to produce rough designs and rough estimates of the various schemes considered. In time the choice of type becomes much more straightforward as one obtains guidance from the rough schemes and the final schemes adopted for earlier work. In nearly every case there is a particular type which will give the most satis-

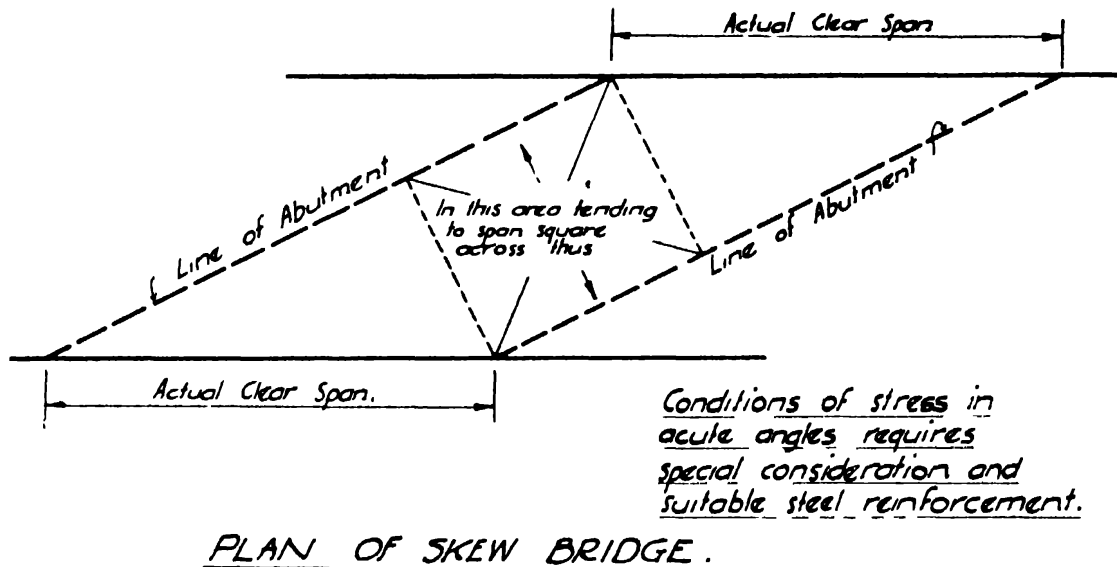


Fig. 19.

factory finished structure for the purpose for which it is built and which is also the most economical to meet the requirements of the case.

In conclusion it should perhaps again be emphasised that this article provides a general indication of the line of thought required in selecting the type of bridge for any site, but that the decision for any particular case depends almost entirely on the site or on the natural and artificial conditions which have been noted.

The Ryburn Dam.

ON September 7 the Wakefield Corporation's mass concrete dam in the Ryburn valley was completed at a cost of about £245,000. The new dam, which was constructed by direct labour to the design and under the supervision of Mr Clemesha Smith, M Inst C.E., the Waterworks Engineer to the Wakefield Corporation, retains water to a maximum depth of 100 ft and is curved on plan to a radius of 700 ft. The maximum height from foundation level to the roadway across the

crest is 121 ft, and the storage capacity is 220,000,000 gallons. At foundation level the maximum width of the base is 84 ft.

The whole of the concrete has been placed by gravity through 10-in. sheet iron tubes, and the depth of the cut-off trench has been reduced by extensive use of cementation.

Illustrated articles describing the design and construction of the dam were published in this journal in July and August, 1930.

Promenade Development at Wallasey.

IN the year 1927 Parliamentary sanction was obtained for the extension of New Brighton promenade to a point 300 yd west of Harrison Drive, at an estimated cost of £900,000. The work now in hand forms the first portion of the scheme designed by Mr I. St. G. Wilkinson M Inst C.E., the Borough Engineer and consists of the extension of the promenade to the Red Noscs, a distance of about

Valves are also provided to enable the lake to be filled on smaller tides. The remainder of the enclosed space is being laid out as promenades, gardens, a motor park, and an open air swimming bath.

The Sea Wall.

Work on the new sea wall was commenced in April, 1931 and this and the marine lake are now completed. The

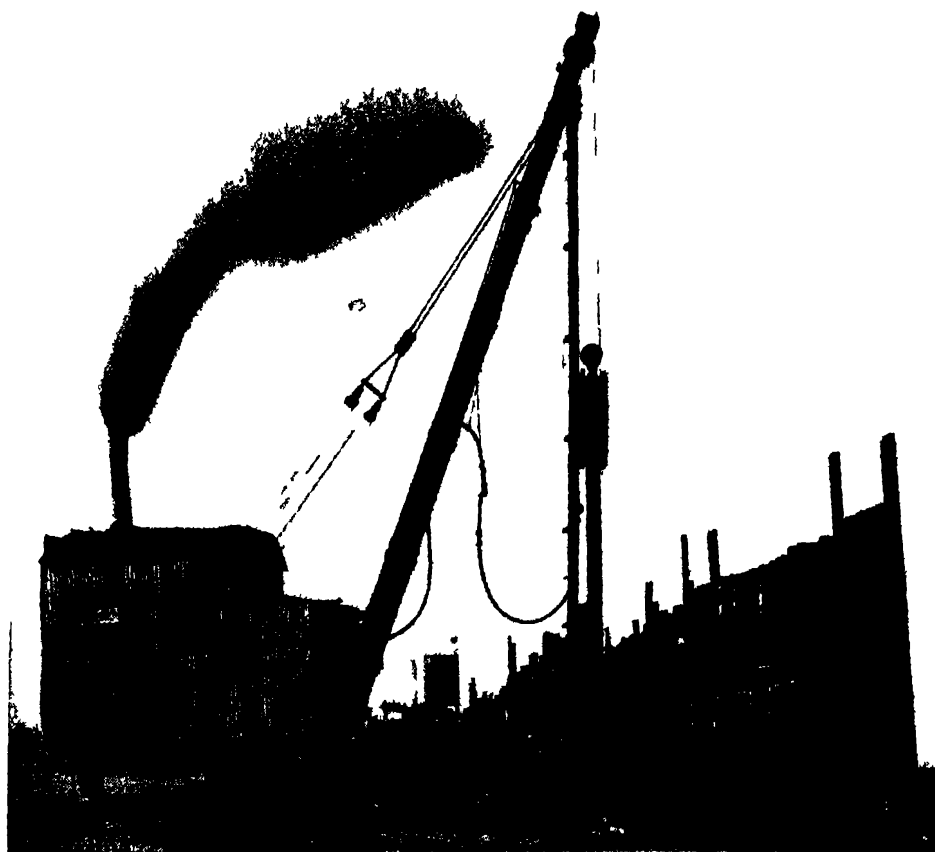


Fig. 1.

$\frac{1}{2}$ mile, and the provision of a marine lake, open-air swimming bath, and public gardens, at a cost of about £500,000. The contractors are Messrs Edmund Nuttall, Sons & Co., Ltd.

An area of 40 acres is enclosed and reclaimed from the foreshore by a new wall containing about 50,000 cu yd of mass concrete. Ten acres are used as a boating lake 4 ft 6 in deep, which is filled by spring tides lapping over the lake wall.

Wall is constructed of mass concrete 16 ft thick at the base, and the total height from foundation to coping level is 38 ft 8 in at the deepest point, one half of this height being below shore level. The foundations are partly on clay and partly on the red Triassic sandstone. In each case the wall was constructed inside a cofferdam of Larssen No. 2 steel sheet piling driven below foundation level and sufficiently high to exclude the tides. At

the deepest section the piles were 40 ft long. A No 7 McKiernan-Terry steam hammer, slung from the jib of a crane mounted on caterpillar track (*Fig. 1*), was used for driving the piles, two piles being driven simultaneously. Compressed-air jetting was used to assist in driving the sheet piles, and proved a very successful method of increasing the rate of penetration. The pressure of compressed air used in driving the steel sheet piling was 90 lb per square inch. A 1½-in. diameter pipe and a ¾-in nozzle were used. The compressed air acting in conjunction with the water in the sand through which the piles

On the land side the wall thickness decreases by stepping back 6 in. at vertical intervals of 2 ft., while the sea face is stepped as shown in *Fig. 2*, the profile being designed to break the impact of the waves, return the water to the sea, and keep the promenade free from water other than spray blown over by the wind

Material excavated from the cofferdam was tipped behind the wall. Additional filling was obtained from excavation of the sandhills by a Ruston-Bucyrus navyy on caterpillar track. The total volume of filling required for bringing the promen-



Fig. 2.

were being driven was found to reduce the average time required for driving a pile from 35 minutes to 25 minutes.

Where the wall foundations were on rock the steel sheet piling was withdrawn, but where the foundation was clay the lower portions of the piles were left in position and 14-in. square precast reinforced concrete piles were driven at 10-ft. longitudinal centres under the toe of the wall to obtain a greater degree of stability. The reinforced concrete piles were driven to a set of 1 in. for 10 blows of a 50-cwt. drop hammer falling 3 ft. The number of piles required was 370, having a total length of 5,760 ft.

ades to the required level was approximately 280,000 cb. yd.

On the sea face and on the stepped back of the walls "Blawforms" were used as shuttering, with a special steel form for the curved nosing of the coping; this was supported by curved steel tee-irons and wedges as shown in *Fig. 3*. Normally the shuttering was supported from the timbering of the cofferdam, but on a short length where the cofferdam was damaged by very heavy seas it was necessary to erect rolled steel joists as soldiers (*Fig. 3*), strutted and bolted in pairs to carry the wall forms.

The mass concrete (quality A) used in

constructing the sea wall is in the proportions of 112 lb of rapid hardening Portland cement to $3\frac{1}{2}$ cb ft of Rossett sand and 7 cb ft of Penmaenmawr granite passing through a $2\frac{1}{2}$ in screen. This is faced on exposed surfaces with a 12 in thickness of quality B concrete in the proportions of 112 lb of rapid hardening Portland cement to $2\frac{1}{2}$ cb ft of sand and 5 parts of granite passing a $1\frac{1}{2}$ in screen. A shutter was not used to separate the two qualities of concrete.

The method adopted to separate the two concretes as they were placed was to place a 12 in layer of B quality

The Blaw-Knox plant used on these contracts is illustrated in Fig 4. This comprised a steel bin batching plant with a 1 cb yd Rex mixer. Concrete was delivered into 1 cb yd skips with bottom opening doors and detachable bodies and transported to the forms in trains hauled by petrol locos. The output of the concreting plant when working to its full capacity is 60 cb yd hourly.

In the sea wall there are plain butt expansion joints at intervals of 60 ft. The joints in the coping are spaced 40 ft apart.

At intervals of 20 ft along the inside of the wall are partition walls 2 ft wide and



Fig 3

concrete adjoining the face of the wall and let this stand at its natural angle of repose following this up immediately with A concrete for the remaining width of the wall. The whole of this concrete was then well punned and the process repeated in 12 in layers until a total lift of 4 ft was attained. For the reinforced concrete work, including the precast piles, the proportions used are 112 lb of rapid hardening cement to $2\frac{1}{2}$ parts of Rossett sand and 5 parts of $\frac{3}{4}$ -in Penmaenmawr granite. In all concrete work the fine aggregate is a pit sand which is washed twice before being delivered on the site.

Above the promenade level these form spaces in which seats are being provided and anchored to 3 in square timber plugs 12 in long cast in the concrete. Seven 12 in diameter outlets with flap and reflux valves are provided through the wall at a depth of 4 ft below the promenade level. In addition the filling is drained by 9 in glazed earthenware open jointed pipes.

The promenade includes a 50 ft and a 24 ft reinforced concrete carriageway separated by an intermediate footpath 10 ft wide. In addition there are a 30 ft footpath between the sea wall and the 24-ft road and a 13 ft path alongside the

50 ft roadway. The promenades are built on filling covered with 9 in. of rock rubble. The carriageways are composed of 8 in. of concrete with B.R.C. reinforcement and surfaced with a 2 in. layer of asphalt and their total area is 28 000 sq yd. Precast concrete kerbs supplied by the Liverpool Artificial Stone Co. Ltd. are being used and the footpaths are surfaced with concrete slabs supplied by Constone Ltd. Of the latter there are 5 000 sq yd made with grey concrete and 22 000 sq yd tinted red. The sub-contractors for the kerb laying and

The site for the bath is 510 ft long by 410 ft wide. On the south side of the bath is the championship area having a length of 165 ft and a width of 60 ft. In this area the minimum depth of water is 5 ft 6 in. at the east and west sides and a minimum depth of 15 ft is maintained in the middle of the area over a width of 33 ft for 10 metre diving. To the north of the championship area the side walls are returned and the width of the bath is increased to 330 ft. The depth of water in a 60 ft width on this length decreases uniformly from 5 ft at the southern edge



Fig. 4.

footpath construction are Messrs John McGeoch & Sons, of Wallasey.

The Swimming Bath.

The open air swimming bath will be completed in May 1934, and will provide accommodation for 2 000 bathers and 7,000 spectators. Among the modern features to be included are a sun bathing beach, under-water and flood lighting, and a deep water championship area. The bath will be 330 ft long by 225 ft wide, and filtration plant will be installed capable of dealing with the whole contents of the bath in a period of eight hours.

to 3 ft 6 in. at the north. Beyond this strip the water level decreases to zero on a width of 80 ft measured on the centre line of the bath. The artificial beach has a maximum width of 25 ft. The total water area is 52,400 sq ft and the contents of the bath are 1½ million gallons.

The water supply will be obtained through two 21 in. cast iron pipes from the marine lake which will act as a settling tank, and after passing through the filters the water will be pumped through concrete lined cast-iron mains with four 9-in. branches leading to the bath inlets. A 12-in. branch off these mains will feed an ornamental cascade. The return from

the bath to the filters will be through a 24 in concrete lined cast-iron pipe

Construction of the Bath.

The bath has been designed with mass concrete walls and a reinforced concrete two course floor the bottom slab of which is laid without expansion joints except where it meets the side wall. This slab rests on a 6 in consolidated layer of rubble and is 6 in thick for a width of 105 ft extending from the northern edge of the bath towards the deep end. For the remainder of the slab the thickness is 12 in. The inner side of the wall is formed with a projecting footing between which and the floor slab is a 1 in thick vertical asphalt expansion joint. A strip of asphalt 2 ft wide covers the junction of the lower slab and the projection of the wall footing. In both slabs the reinforcement consists of B R C fabric double reinforcement being used in the 9 in upper slab and two layers of single reinforcement in the bottom slab where the latter is 12 in thick. Before placing the top slab the upper surface of the bottom course is painted with bitumen.

In the upper slab two series of expansion joints have been formed dividing the slab into rectangular panels approximately 50 ft by 30 ft in area. These joints consist of a 2 ft wide horizontal strip of asphalt 1 in thick and a central 1 in vertical thickness extending through out the depth of the 9 in slab. The floor is treated with a rendering of water proofed cement and a finishing coat of white Portland cement, with a rough non slip surface.

The bath wall is founded partly on clay

and partly on sand, the portion surrounding the deep part of the bath resting on forty 14-in square precast reinforced concrete piles driven at from 12-ft to 13-ft centres. In the piled length of the wall the heads of the piles are stripped for a length of 2 ft in order to provide a bond between the reinforcement and the mass concrete of the wall. The minimum thickness of the wall is 1 ft 6 in and increases by steps 10 in wide and 2 ft deep. The wall footing is 12 in thick. At the section on the centre line at the deep end the wall is 17 ft high. A 12 in by 3 in cast stone non slip coping is provided at the edge, and the faience scum channel is fitted with outlets at 30 ft centres to a stoneware drain. The face of the wall is rendered with white water proof cement.

The bath surround is a 6 in reinforced concrete slab with an indented surface, laid to fall to a 12 in by 1 in surface channel discharging at gullies at 50 ft centres.

The mass concrete in the bath wall is of the richer mixes previously mentioned, except in the deep water diving area where quality A concrete faced with 12 in of quality B was used. Expansion joints in the bath wall are six in number, and are formed of asphalt 1 in thick, the front edge of each joint being caulked with lead wool. Concrete is placed in the floor by means of rectangular 1 cb yd bottom opening skips hauled on jubilee track from the central mixing station and unloaded by a crane travelling on caterpillar track. The contract for the bath buildings has recently been let to Messrs Wm Tomkinson of Liverpool.

British Standards Institution.

MR MAURICE F G WILSON, M Inst C E, who has been chairman of the Main Committee and chairman of the General Council of the British Standards Institution for the past five years, has retired in accordance with the by-laws, and Mr E J Elford, M Inst C E, Borough Engineer and Surveyor of Wandsworth, has been elected to succeed him as chairman of the General Council for the next twelve months.

Combined Bending and Thrust or Tension.

By A. G. BOORMAN, D.I.C., M.Sc., A.M.Inst.C.E.

THE rapid selection of concrete sections to resist combined bending stresses and compression or tension presents some difficulties. It is believed that the following method of analysis will be found convenient, particularly when the bending moment does not depend upon the section chosen, it can also be used in other cases without difficulty, whether the load is tensile or compressive, and whether the whole section or only a small portion of the concrete is in compression.

(Considering the case of such a section in which thrust and bending are combined (Fig. 1) and making the usual assumptions, it may readily be shown that

$$P = bdc \left[\frac{n'}{2} + p'(m-1) \frac{n'-g'}{n'} - pm \frac{1-n'}{n'} \right].$$

Assuming p , p' , g' and m to be constant, and n' constant for the time being

$$P = \alpha bdc.$$

The equation remains the same in form when t is the limiting factor.

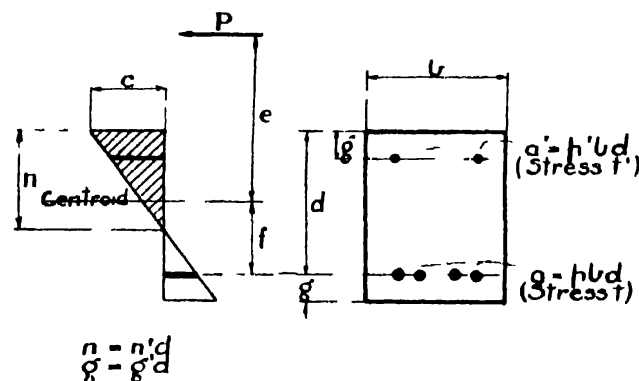


Fig. 1.

Taking moments about the steel in tension

$$M' = bd^2c \left[\frac{n'}{2} \left(1 - \frac{n'}{3} \right) + p' \left(\frac{n'-g'}{n'} \right) (m-1)(1-g') \right] \\ = \beta bd^2c.$$

This is not to be equated to the external moment, for the latter must be taken about the centroid given by the equivalent area of the steel and the actual area of concrete in compression for any particular position of the neutral axis. This is, at first, a little difficult to visualise and has been overlooked in some analyses, but it may be illustrated as follows. Only the concrete in compression exerts a resistance, therefore that area only above the neutral axis is effective in fixing the centroid. The centroid is the position in which the resultant resisting load would lie if uniform strain could exist over the whole area and this portion of the

* This becomes $(m-1)$ when the neutral axis falls below the lower steel.

concrete only resist stress The true moment to which the external moment must be equated is

$$M = M' + Pf,$$

where f is the distance of the centroid from the tension steel and depends on the position of the neutral axis. However, it will be found that

$$f = \frac{d \left[n' \frac{n'^2}{2} + (m-1)p'(1-g') \right]}{mp + n' + (m-1)p'} = \gamma d$$

Then

$$\begin{aligned} M &= \beta b d^2 c + \gamma b d c \gamma d \\ &= b d^2 c (\beta + \gamma^2) \\ &= \delta b d^2 c \text{ say} \end{aligned}$$

If therefore, there are two different concrete sections so loaded that n' and p , p' and g' are the same then for a given limiting value of c the loads taken will be proportional to the dimensions of the section. Thus

$$\frac{P_1}{P_2} = \frac{b_1 d_1}{b_2 d_2} \text{ and } \frac{M_1}{M_2} = \frac{b_1 d_1^2}{b_2 d_2^2}$$

Provided n' is the same for both its value is immaterial

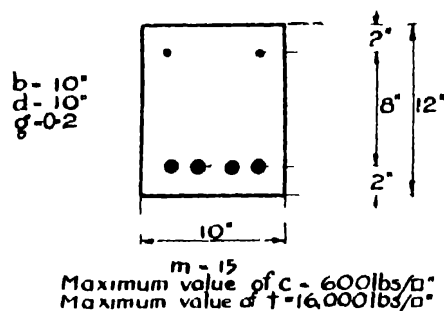


Fig. 2.

It may be shown that this relation holds when compression exists over the whole section, or when a tension is combined with a bending moment.

A rectangular concrete section, called a "reference section," may be chosen, and the values of the external load and external moment calculated which may co-exist for all values of n' to give stresses that do not exceed the values of c or t selected. Any other rectangular section of whatever dimensions, but having steel percentages and the value of g' equal to those of the reference section, may be called a "similar section," but will not generally be geometrically similar. From the known properties of the reference section the properties of a similar section may be obtained by a single simple mathematical operation.

As an example a reference section may be taken with the dimensions shown in Fig. 2. Here $b = 10 \text{ in}$, $d = 10 \text{ in}$, $g' = 0.2$, $m = 15$, and the maximum values of c and t are 600 lb per square inch and 16,000 lb per square inch respectively. These particular values need not be taken, but reference curves may be worked out for any figures which appeal to the designer. The preparation of the curves will be found a little laborious, but once obtained will be equally useful to those given.

In Figs. 3, 4 and 5 are curves derived for the reference section, using various percentages of steel from 0.2 per cent. to 3.0 per cent. on the tension side and for the following conditions:

(a) Steel on the tension side only,
 (b) Steel on the tension side equal to twice the steel area on the compression side, and

(c) Steel on the tension side equal to the steel on the compression side.

It may be found convenient to re-draw these in three different colours on

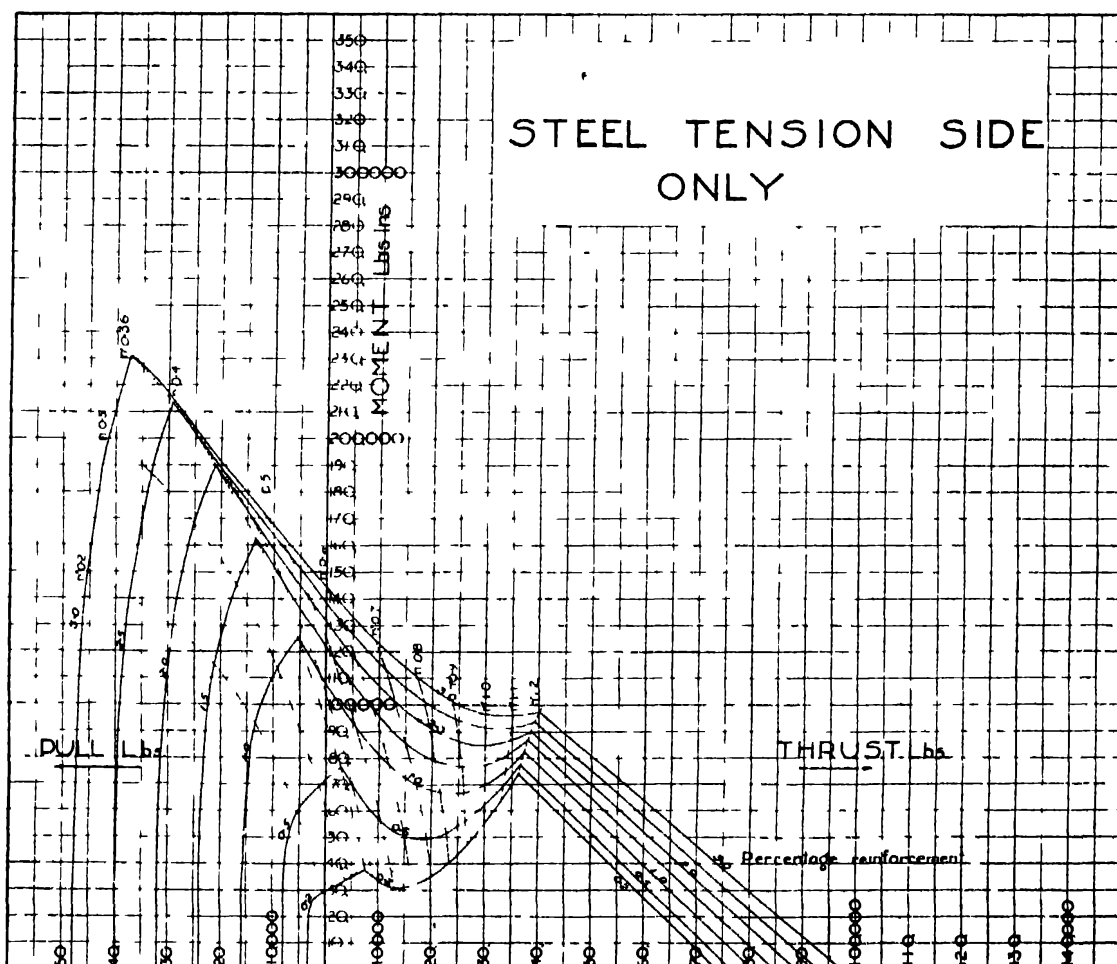


Fig. 3.

the same plan and to a larger scale, when it will not be difficult to interpolate both for steel percentages and for compression-to-tension steel ratios up to unity.

It will be observed that the method of plotting shows in a single diagram any combination of tension or thrust with bending moment, whether compression exists over the whole depth of the section or on part of it only. For sections having compression steel the curves have been stopped where the neutral axis has moved up to the steel line. In general, the position of the neutral axis should be noted when selecting a section.

The left-hand peak for each curve shows the point where the steel stress ceases to be the limiting factor (which is the case to the left of the peak), or where

the concrete stress ceases to be the deciding factor (which is the case to the right) For a small range of combinations only is it possible to choose a section in which both tension steel and concrete are simultaneously fully loaded The right-hand peak is the position where the whole area of concrete is in compression, as is the case also to the right of this point All curves are continuous through the moment axis This may be understood by considering the moment for low loads to be

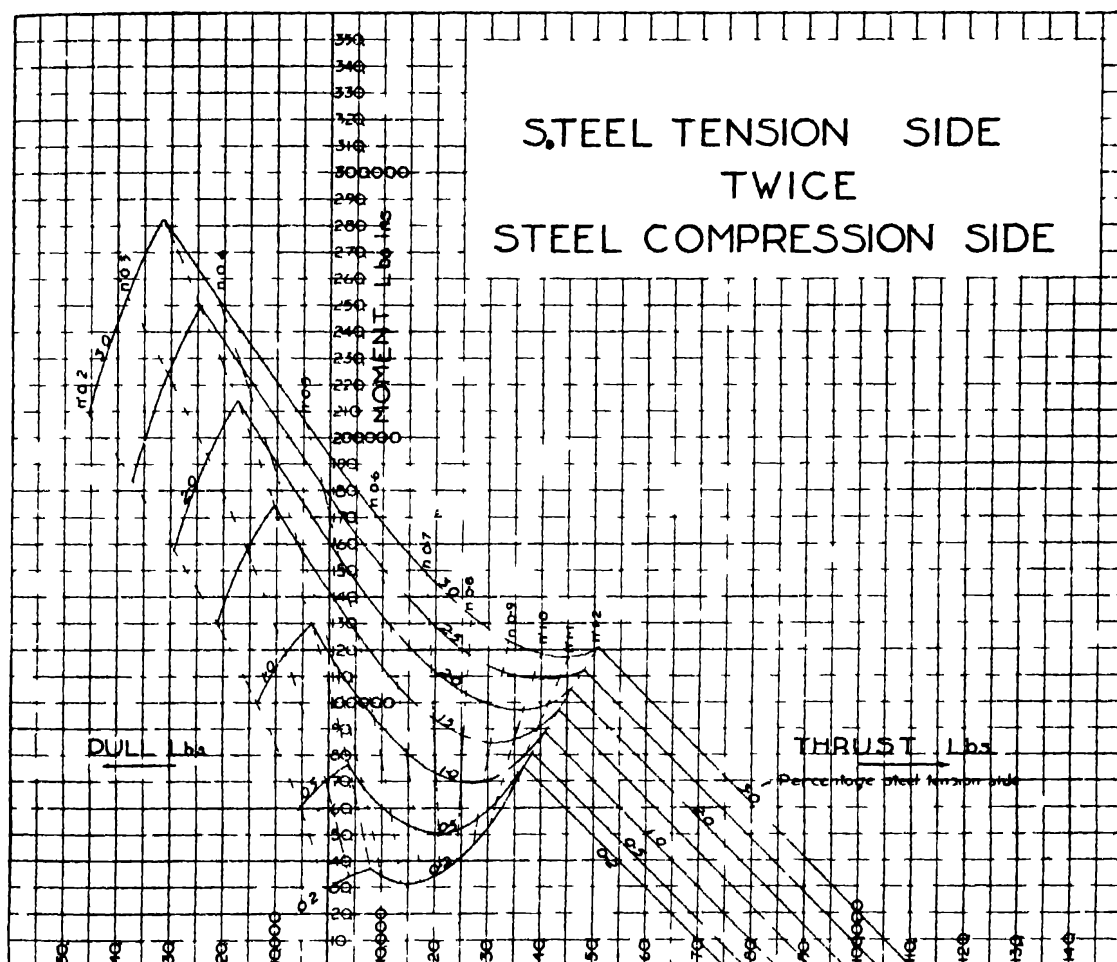


Fig 4

produced by a very small thrust acting at a very great distance The load gradually reduces, moving to an infinite distance, so that a finite moment is produced with zero load The thrust then changes sign to a very small tension and gradually returns, producing greater moments for gradually increasing loads

It must be noted that the whole of the section of reference has been used in calculating resisting forces and bending moments Where this is contrary to local regulations for certain members a set of curves may be calculated with this variation, but since the neglect of the concrete outside the steel only affects the side of the member which is in compression, and since here the steel will not be highly stressed, a close approximation may be obtained by using the reference curves to choose the section and then adding the thickness of fireproofing to the

dimensions. The steel in compression may then be moved outwards a small distance in order to make use of its full value.

The following example will assist to demonstrate the method.

To choose a section to carry 200,000 lb thrust, and 2,000,000 lb in bending moment assuming $c = 600$ lb per square inch, $t = 16,000$ lb per square inch, and $m = 15$.

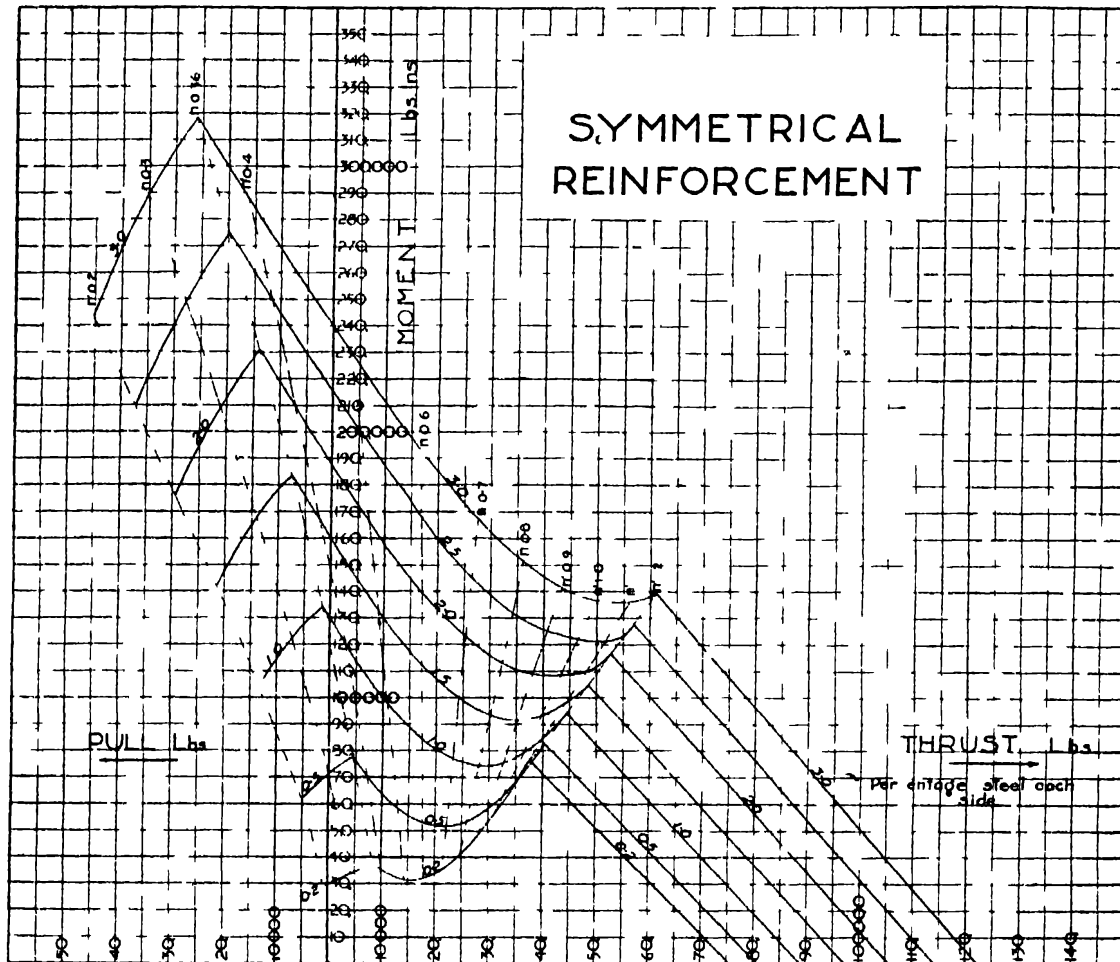


Fig. 5.

Try a section with $d = 30$ in and $b = 20$ in

Then the equivalent load (that producing equal stresses) on the reference section would be

$$P = \frac{200,000}{2 \times 3} = 33,300 \text{ lb}$$

$$M = \frac{2,000,000}{2 \times 3^2} = 111,000 \text{ lb-in}$$

From inspection of the reference curves it is evident that the section would suit if reinforced with 2 per cent on both sides, or with 2.5 per cent on the tension side and 1.25 per cent on the compression side, or with more than 3 per cent. on one side only (the latter being unsatisfactory).

Assuming a section with $d = 35$ in. and $b = 15$ in., the equivalent loads become

$$P = \frac{200,000}{1.5 \times 3.5} = 38,100 \text{ lb.}$$

$$M = \frac{2,000,000}{1.5 \times 3.5^2} = 109,000 \text{ lb. in.}$$

The following reinforcement will be suitable: (a) 2 per cent. on both sides, (b) 2.5 per cent. on the tension side and 1.25 per cent. on the compression side, or (c) more than 3 per cent. on one side only (the latter being unsatisfactory).

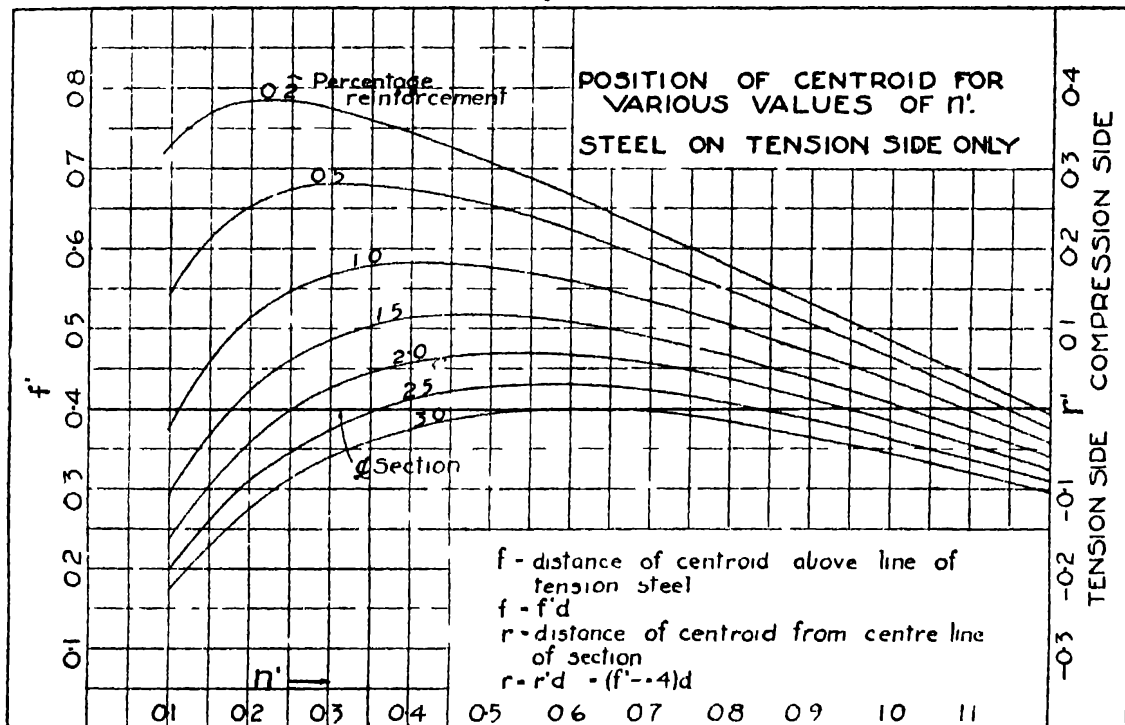


Fig. 6.

In this way different sections may be compared rapidly. Further, it is possible to use the curves to visualise the effect of variation in the external moment or load or in both.

Thus, for the case considered, choosing a 30-in. by 20-in. section reinforced with 2 per cent. of steel on both sides, if the thrust can fall to zero the section will be perfectly safe, even with a 70 per cent. greater bending moment. If the thrust is constant, any increase in bending moment will cause the safe stress in the concrete to be exceeded. If the bending moment remains constant, an increase in the load will only cause a small overstress in the concrete, even for a 60 per cent. increase in load.

For low percentages of reinforcement it must be noted that for sections chosen for low eccentricities, it is possible to render the section dangerous to the point of fracture by reduction of the thrust if unaccompanied by reduction in the bending moment. A similar effect is obtained if it is possible to reduce the load

without moment for sections subject to tension and bending, but this applies in the use of high reinforcement percentages.

Case where the eccentricity is known.

This occurs where the load to be carried is eccentric, either fixed in a certain position or necessarily at a certain distance outside the stanchion.

In the first case, if it is possible to move the stanchion so as to carry the load centrally, this should be done. If the eccentricity is very low (less than about $0.16d$) compression will exist over the whole surface, and the centroid will be fixed for a fixed section, and will always be central for a symmetrically reinforced

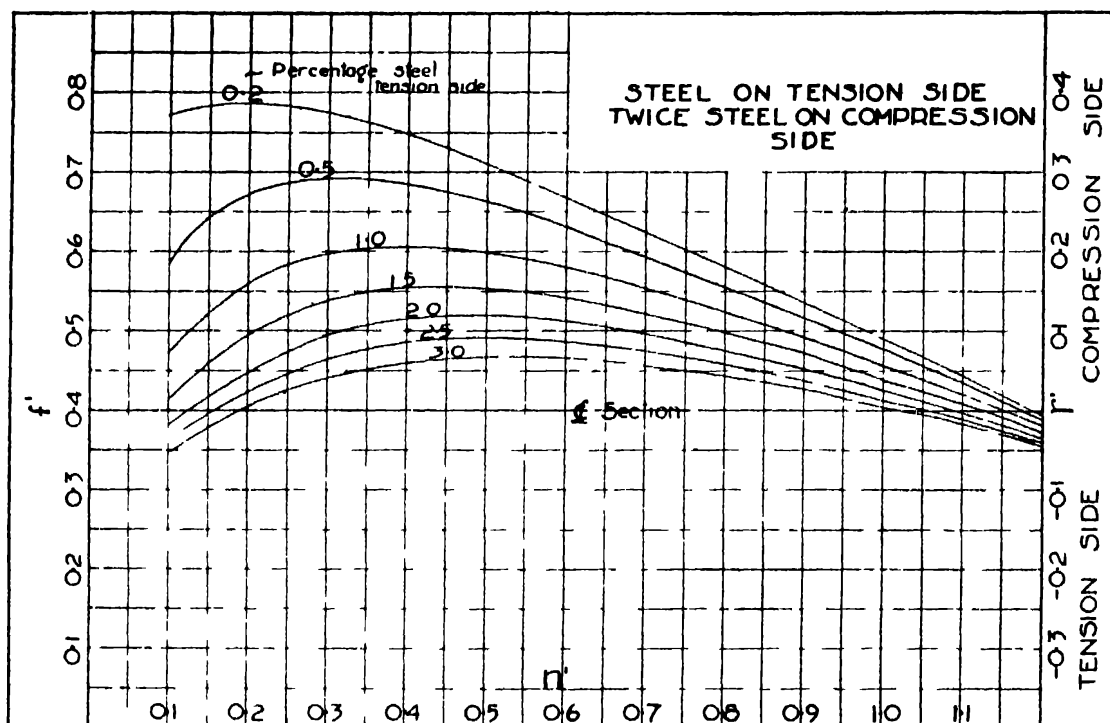


Fig. 7.

member. Thus choice of a trial section will immediately fix the eccentricity and therefore the bending moment, and no difficulty occurs.

In the second case the bending moment depends on the properties of the section; consequently for a first trial a position for the centroid must be assumed. In the case of thrust, if it is desired to use less than 2 per cent. of reinforcement, take the centroid at a distance from the centre line of the member towards the compression side of $0.1d$ when the thrust is within $\frac{1}{4}d$ of the tension steel, and at $0.2d$ when the thrust is farther than $\frac{1}{4}d$ from the tension steel.

If it is desired to have more than 2 per cent. of reinforcement, use $0.1d$. In the case of tension (omitting instances where there is less than 1 per cent. of steel) take the centroid at a distance of $0.1d$ from the centre line of the member towards the compression side. Then the first trial section will fix the bending moment, and the curves may be used.

When suitable dimensions have been obtained, note the value of n' and,

from the centroid curves given in *Figs. 6, 7, and 8*, find the actual position of centroid. If this agrees with the assumption, the first trial is correct. If not, take the new position of the centroid and obtain the corresponding bending moment, and repeat.

Example.

It is desired to carry a load of 200,000 lb. from a bracket at a distance of 4 in. from the edge of the stanchion.

Try a section with $d = 30$ in. (overall measurement 36 in.) and $b = 24$ in. The eccentricity from the centroid is 18 in. + 4 in. - 3 in. = 19 in. (assuming that the centroid is at 0.1d from the centre line on the compression side of the

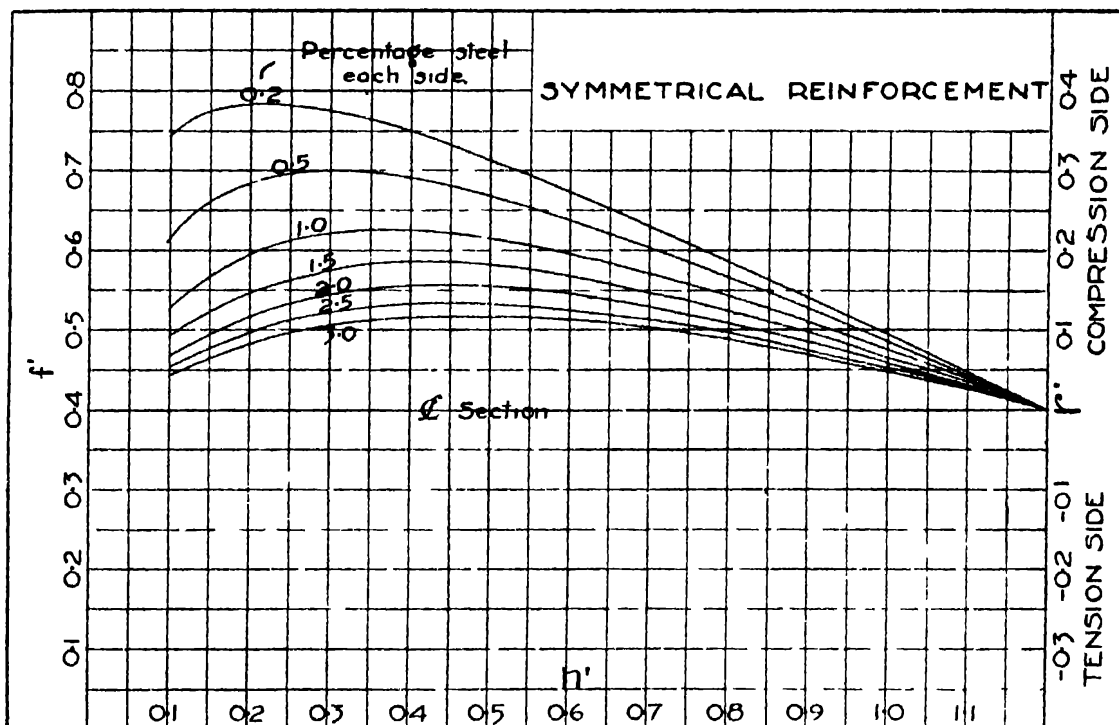


Fig. 8.

stanchion) and the bending moment is $19 \times 200,000 = 3,800,000$ lb. in. The equivalent loads on the reference section are

$$P = \frac{200,000}{3 \times 2.4} = 27,800 \text{ lb.}$$

$$M = \frac{3,800,000}{3^2 \times 2.4} = 176,000 \text{ lb. in.}$$

This requires more than 3 per cent. on both sides.

If we increase the width to 28 in.,

$$P = \frac{200,000}{3 \times 2.8} = 23,800 \text{ lb.}$$

$$M = \frac{3,800,000}{3^2 \times 2.8} = 151,000 \text{ lb. in.}$$

2.5 per cent. on both sides is just sufficient, and $n' = 0.67$, $r' = 0.118$, and $r = 0.118 \times 30 = 3.54$.

The actual eccentricity is $18 + 4 - 3.54 = 18.46$ in. and the actual bending moment is $18.46 \times 200,000 = 3,690,000$ lb. in.

The bending moment being somewhat smaller than in the first trial, a 27-in. section width is tried.

The equivalent loads become

$$P = \frac{200,000}{3 \times 2.7} = 24,700 \text{ lb.}$$

$$M = \frac{3,690,000}{3^2 \times 2.7} = 152,000 \text{ lb. in.}$$

The percentage reinforcement for this width would require to be a little greater than 2.5.

It will be clear that if a square or flat section were desired, the first trial section would have been chosen accordingly; the process can be repeated to give any required shape.

Tests of Vibrated Concrete.

At the University of Stuttgart comparative tests of vibrated and hand-tamped concretes with various water-cement-ratios have recently been carried out by O Graff and K. Walz. The vibration was produced by a "Pervibrator"—described in this journal in December 1930—a machine which floats in the freshly-placed concrete. The following notes on the tests have been compiled from an article in "Beton und Eisen."

Tests were made on columns approximately 12 in. square and 4 ft. 2 in. long and on 12-in. cubes. Two series of specimens were made, one containing about 400 lb. of Portland cement per cubic yard of finished concrete and the second containing 260 lb. per cubic yard. The aggregates used were Rhine sand and gravel.

In the first series the proportions of the dry volumes of aggregates to 1 part of cement were 1.60 parts from zero to 1 mm., 0.80 parts from zero to 3 mm., 1.60 parts from 3 mm. to 7 mm., and 4 parts from 7 mm. to 30 mm. This series was sub-divided into two parts, A and B, by using different quantities of gauging water, the water-cement-ratios being 0.52 and 0.81 by weight respec-

tively and the corresponding weights of cement in 1 cb. yd. of concrete 400 lb. and 401 lb. respectively.

The proportions of the volumes of dry aggregates in the second series were 2.60 parts from zero to 1 mm., 1.30 parts from zero to 3 mm., 2.60 parts from 3 mm. to 7 mm., and 6.50 parts from 7 mm. to 30 mm. Two sub-divisions, C and D, were made with cement-water-ratios of 0.84 and 1.29 by weight and containing 260 and 255 lb. of cement per cubic yard.

After being stored for 7 days under damp cloth and 21 days in air the specimens were weighed and tested with the following results. The figures given are the average results for vibrated specimens and those enclosed in brackets are for tamped specimens.

Specimen	Density (lb. per cubic foot)	Compressive strength (lb. per square inch)
Column A . . .	148 (140)	5400 (1960)
Cubes A . . .	(140)	(2470)
Column B . . .	153 (143)	2020 (2180)
Cubes B . . .	(144)	(3180)
Columns C . . .	148 (141)	2600 (1400)
Cubes C . . .	(142)	(2060)
Columns D . . .	146 (141)	738 (880)
Cubes D . . .	(142)	(1350)

The Foundations of the Ford Motor Company's Power House at Dagenham.—III (Concluded).

By R. V. ALLIN, M.Inst.C.E.

Turbine Basement Walls.

THE walls of the basement are of reinforced concrete designed as cantilevers 24 ft. high from basement to ground floor at + 20.00 O.D. (*Figs. 30 and 30A*). To allow a layer of asphalt to be put on at the back of the walls for their full height before the mass concrete was poured, a skin wall was necessary of sufficient thickness to embed the walings of the steel sheeting, which consisted of rolled steel joists. Owing to leakage of water through the sheet piling it was found impracticable to build this wall in concrete sufficiently watertight for satisfactory asphaltting. A 9-in. brick wall backed with concrete was therefore substituted on which the asphalt was laid, drainage being effected as the work proceeded by omitting bricks where necessary. The reinforcement of the main wall was arranged in groups of six bars with intervals between the groups to facilitate the work of asphaltting after the bars had been placed. The thickness of the wall between the face of the asphalt and the back of the 2-in. hollow space behind the 9-in. glazed brick facing is 6 ft. At the corners of the basement there occurs a change in the cantilever action of the walls due to the restraint of the end walls which is considerable, and small-scale experiments were made to determine the distribution of the stresses. *Fig. 31* shows the arrangement of the diagonal reinforcement employed as the result of the investigation.

Pump House and Valve Chamber.

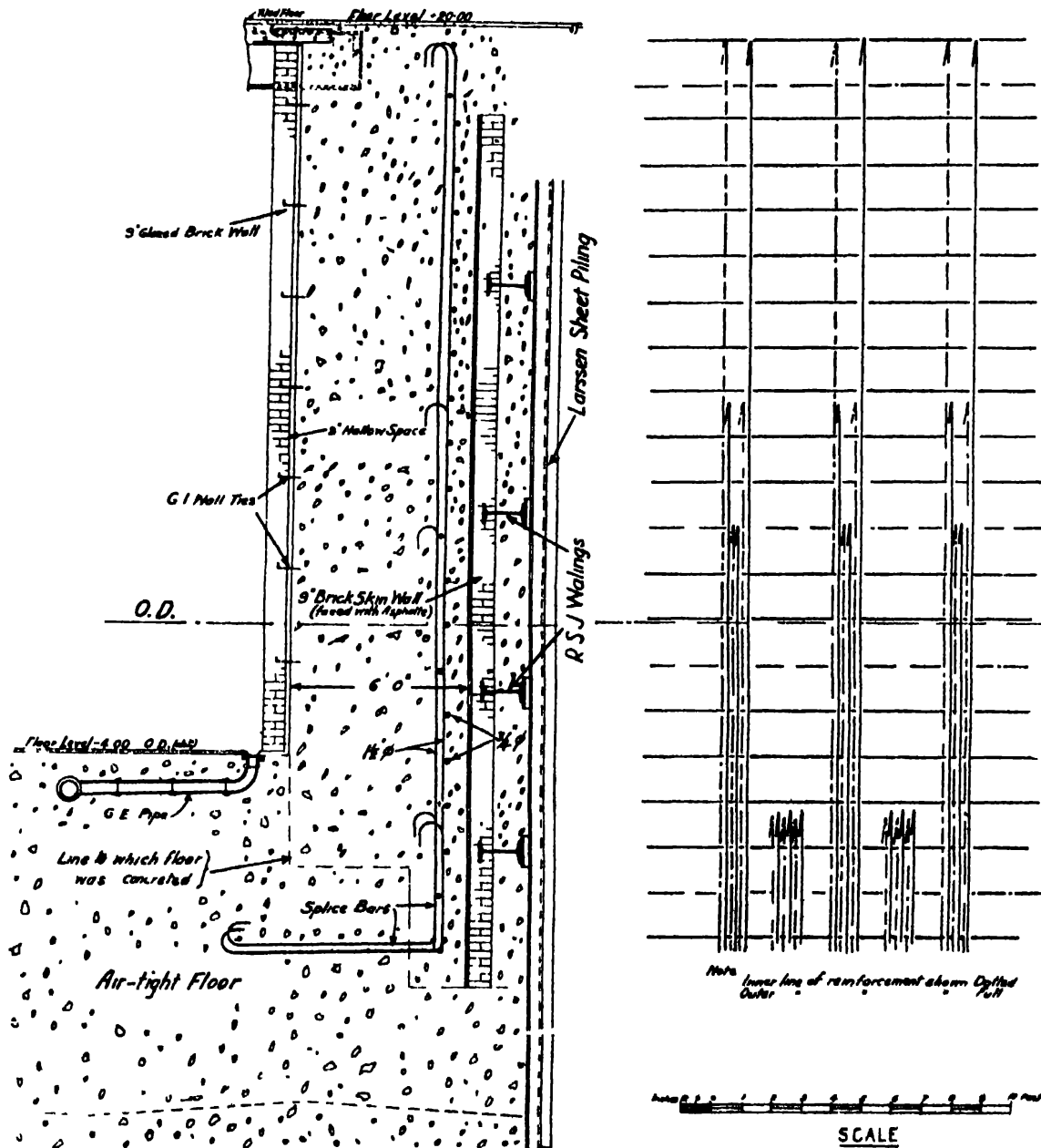
In this part of the basement the loads were such that the floor and walls could be safely carried by piles. The pump well was formed in the lower portion of a caisson 54 ft. long by 25 ft. wide, which was sunk in compressed air to a depth of — 35.25 O.D. and subsequently cut off above floor level (*Fig. 32*). After the caisson had been sunk the piles mentioned were cut down as the excavation around it proceeded. Those piles which had toe levels above that at which the caisson had been founded were re-hammered, and it was found that the material around them had been loosened by the sinking of the caisson to such an extent that some of them went down a further 14 ft. before showing the set to which they had originally been driven.

On the site of the pump well caisson No. 5 Larssen sheet piling, 80 ft. long, forming a part of the original scheme, had been driven. An endeavour was made to pull out this sheeting, but although a pull of 300 tons was applied by jacks, which stretched the pile 3 in. elastically, no permanent withdrawal resulted (*Fig. 33*). It was therefore necessary to burn off this sheeting in short lengths ahead of the cutting edge of the caisson as it was lowered. Both oxy-acetylene and the electric arc processes were used, but the former method proved more satisfactory.

In carrying out the excavation around the pump well caisson considerable trouble was experienced through the escape of air from the faces of the approaching tunnel into the ballast, the top of which was about 3 ft. below the lowest point in the excavation. This air chiefly came in through gaps between the

sheet piling and the outside of the intake shaft, and brought considerable quantities of water with it. Although it was possible to deal with the water, the air caused a considerable amount of trouble when pouring the concrete. This difficulty was met by leaving vent pipes in the concrete as a relief to the air pressure. These pipes were afterwards grouted up.

The chamber which contained the screens was formed in a steel caisson sunk under compressed air to a depth of - 36.25 O.D. There were two air-tight roofs in this caisson, the lower one over the working chamber and that required for constructing the openings into the pump well and intake shaft.



TYPICAL SECTION OF BASEMENT WALL.

ELEVATION OF REINFORCEMENT.

Fig. 30.

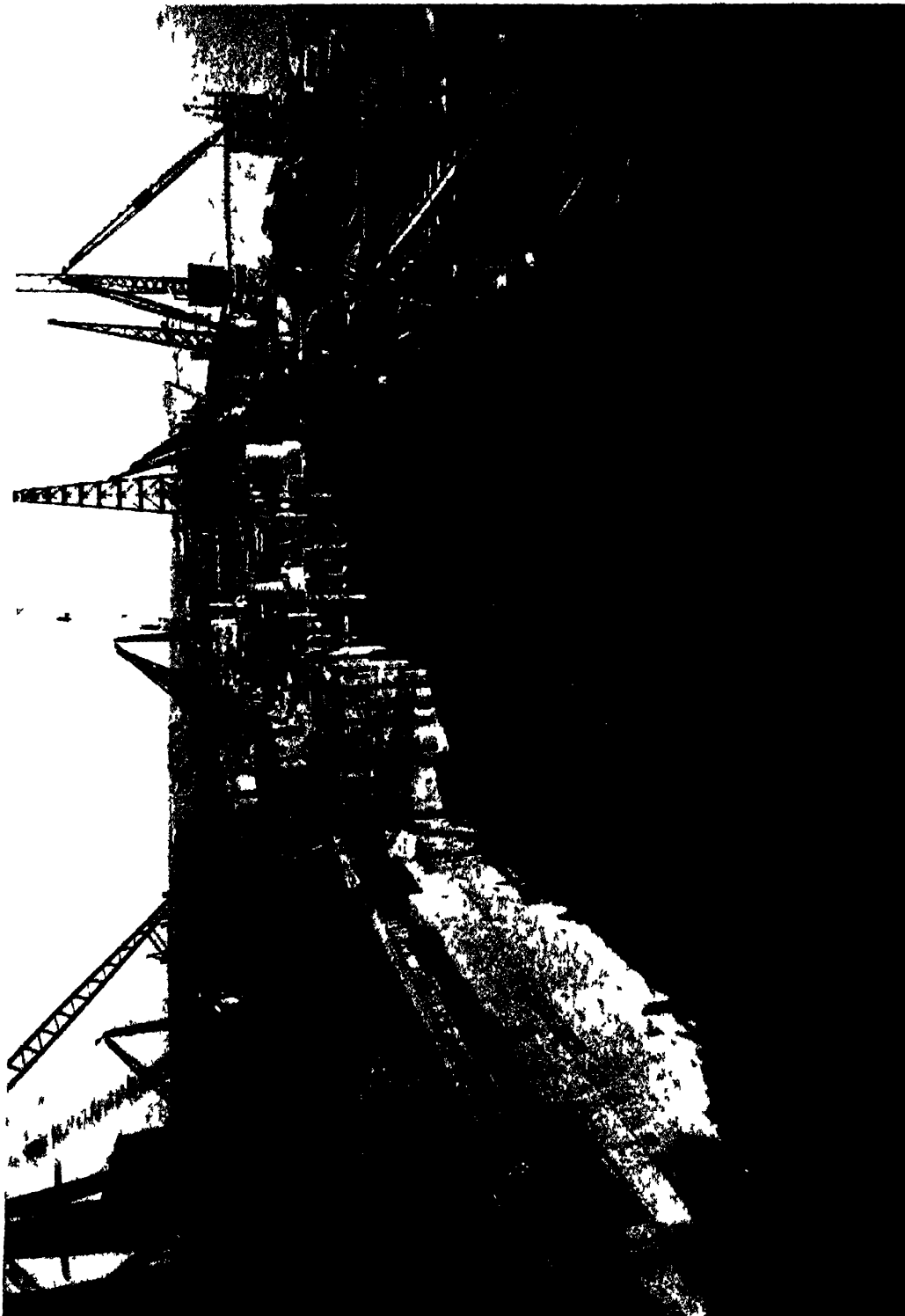


Fig. 30A.

The roofs were constructed of rolled steel joists and concrete, through which the two working shafts passed. When making the openings to the pump well caisson and intake shaft it was discovered that a water channel with a cross sectional area of about 2 sq. ft. had been formed around three sides of the caisson. This channel had been made by the air pressure from the tunnels passing through

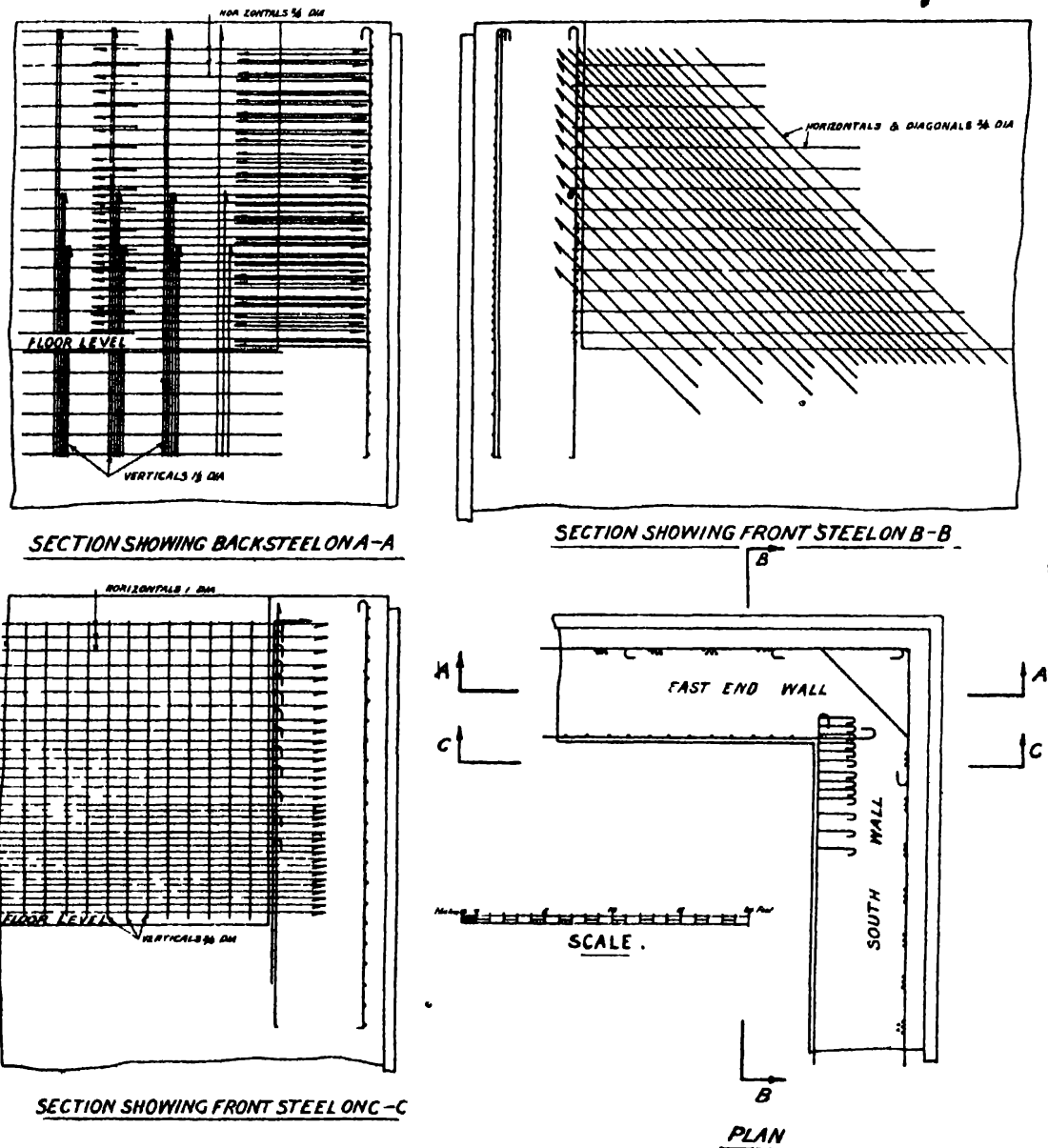


Fig. 31.

the soft peat immediately above the ballast (at about - 17.00 O.D.) into the excavation for the pump well basement mentioned. After sufficient air pressure had been put on to drive the water out of this channel it was filled with lime grout.

The concrete in the north-west wall of the north-west opening from the screen chamber into the intake shaft was found to be permitting a leak to the extent of about 350 gallons per hour at a level of about - 20.00 O.D. This

water, which came from the back of the wall, was under a head of about 20 ft. Messrs. John Mowlem & Co., Ltd., the contractors, successfully applied a chemical consolidation process to seal the voids in the concrete through which the water was passing. Briefly, this process consists in forcing into the concrete under pressure two chemical solutions successively which re-act on each other and form an amalgam which fills the interstices of the concrete and renders it watertight. In this instance six holes of about 2 in. diameter and about 2 ft. apart were drilled into the face of the concrete wherever water was entering



Fig. 32. •

to depths and in directions dictated by experience. Solution No. 1 was pumped in under a pressure of about 300 lb. per square inch—with hand pumps—until it appeared in many places on the face of the wall; it was immediately followed by solution No. 2 at a higher pressure (600 lb. per square inch) to mix the chemicals. These operations were repeated until no signs of leakage appeared over a distance of 24 in. radius around the hole treated, and were continued until complete watertightness was attained.

The screen chamber was divided by concrete walls into four compartments for installation of two twin-flow screens and regulating penstocks.

The two land shafts consisted of ten steel strakes each 13 ft. 6 in. inside diameter. Each shaft had two compressed air working chambers, one for sinking the shaft and one immediately above it, into which the shield was received

at the end of its drive from the river shaft, through a 9-ft. diameter plugged opening. The sinking of the shafts was controlled in the earlier stages by hydraulic jacks supported on guide trestles and attached by links to the steel work (*Fig. 35*). Water kentledge controlled the subsequent sinking. The top of the shafts was at + 19.50 and they were founded at - 49.50 on dark sandy clay. The axes of the tunnels entered the shafts at - 36.00 O.D.

Concrete.

The concrete used on the work described was of the following qualities. Quality A: 112 lb. (1 bag) of rapid-hardening Portland cement to 6 cb. ft. of



Fig. 33.

Thames ballast (not exceeding $2\frac{1}{2}$ in.); Quality B: 112 lb. of rapid-hardening Portland cement to 6 cb. ft. of Thames ballast (not exceeding $1\frac{1}{2}$ in.); Quality C: 112 lb. of rapid-hardening Portland cement to $2\frac{1}{2}$ cb. ft. of sand up to $\frac{1}{4}$ in., and 5 cb. ft. of coarse aggregate ($\frac{1}{4}$ in. to $\frac{3}{4}$ in.); Quality E: 112 lb. of high-alumina cement to $2\frac{1}{2}$ cb. ft. of sand up to $\frac{1}{4}$ in., and 5 cb. ft. of coarse aggregate ($\frac{1}{4}$ in. to $\frac{3}{4}$ in.). Quality A was specified for mass concrete under compressed air floors generally; B for compressed air floors, the walls of the turbine basement, screen chamber and shafts, and pile caps; C for beams and slabs; and Quality E for precast piles.

It was specified that a $\frac{1}{4}$ -cb. yd. sample from each delivery of Thames ballast should be screened and that the quantity of sand passing through a $\frac{1}{4}$ -in. mesh should not be less than 40 per cent. or more than 65 per cent. of the ballast retained on the screen, and that if necessary sand or screened ballast should be added until the necessary proportions were obtained. It was further specified that in the event of the voids in the aggregate—after the addition of the sand

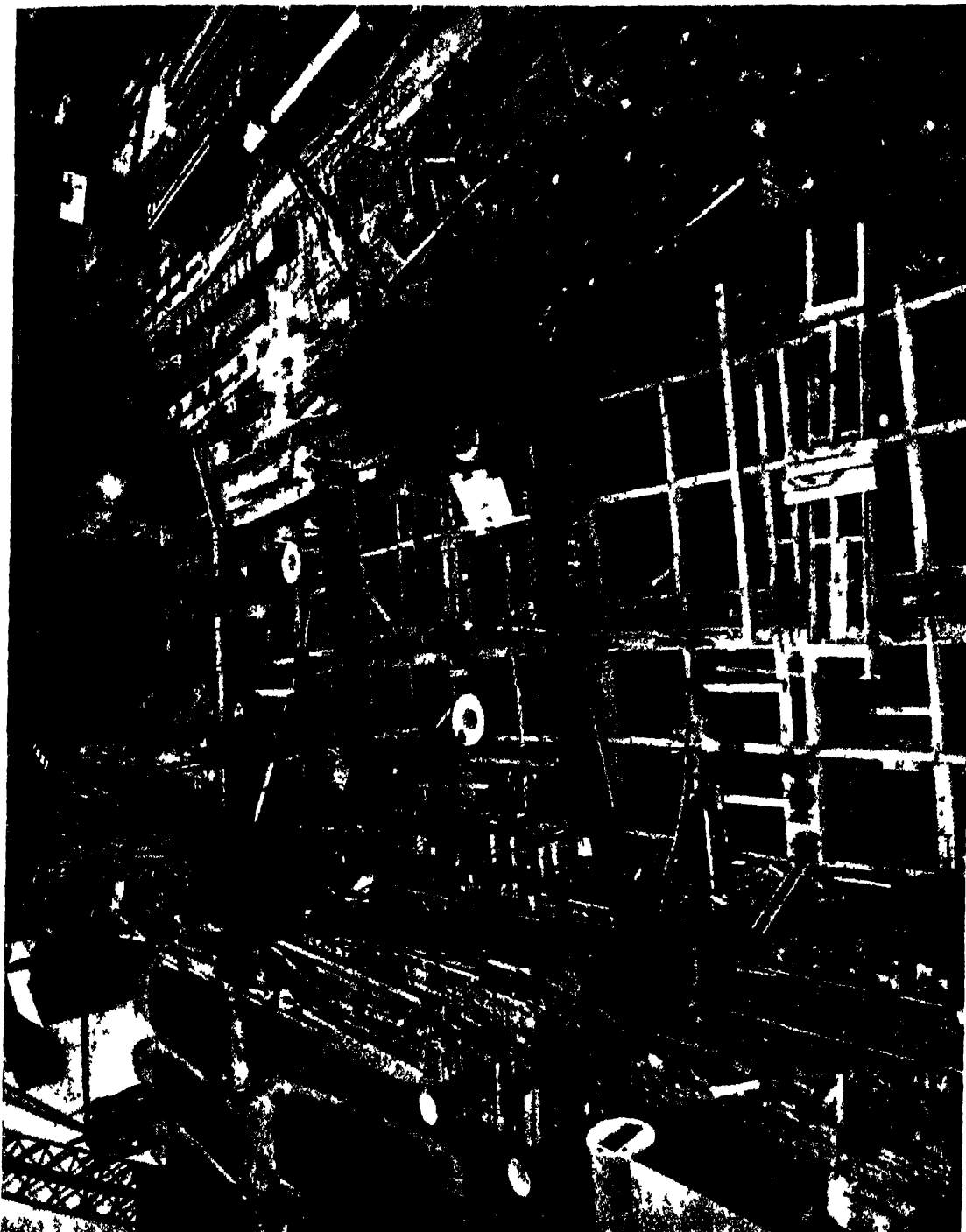


Fig. 34.

required exceeding 15 per cent, the proportion of cement should be increased until it was at least 25 per cent in excess of the voids.

The following minimum crushing strengths for 6 in test cubes, seven days after casting, were specified: Qualities A and B, 2,500 lb per square inch, Quality C, 3,000 lb per square inch, Quality E, 4,000 lb per square inch.

Owing to the fact that there was no direct access by road to the site of the power house during the greater part of the construction, practically all the material and plant had to be water borne, and a timber wharf was built along the whole frontage of the site from which material was handled and placed by

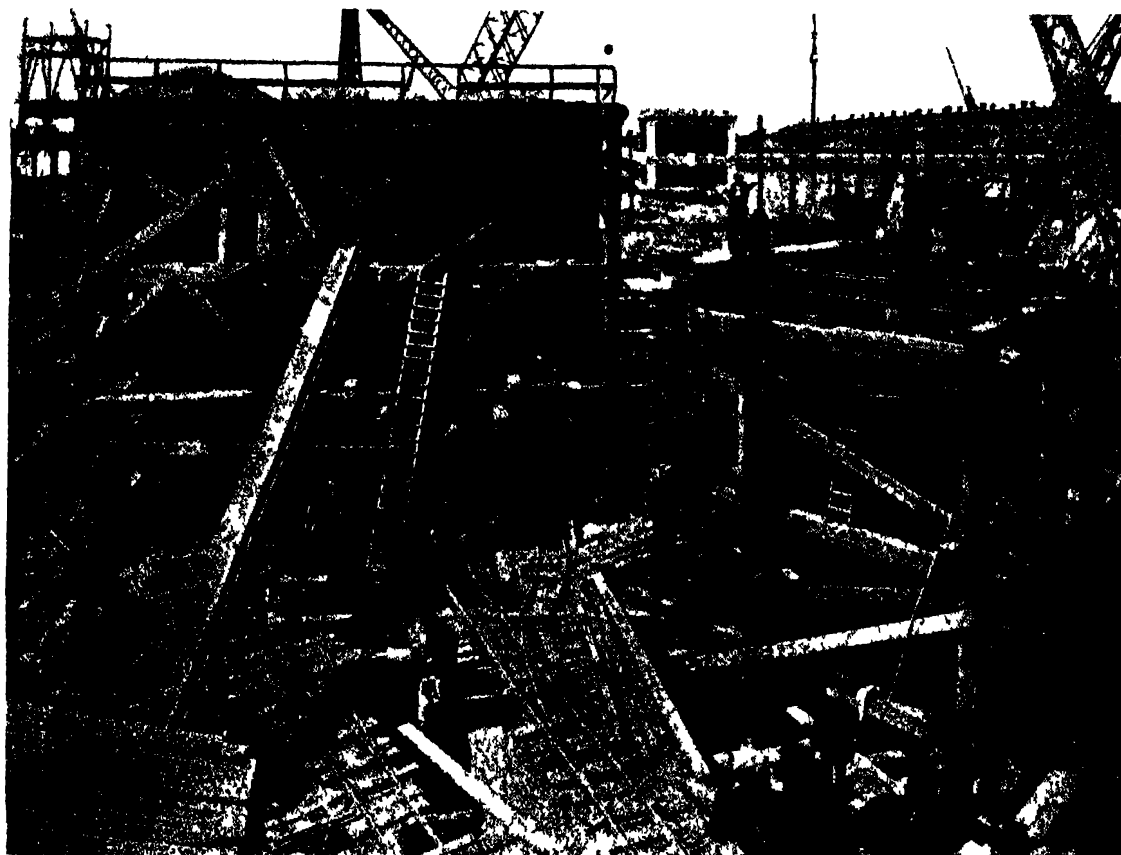


Fig 35

12-ton and 15-ton electric derricks (*Fig 34*). At the west end of this wharf was erected the main concrete mixing station (*Fig 35*) which consisted of three $1\frac{1}{2}$ -cb yd. batch mixers, electrically driven, supplied with aggregate from overhead hoppers from a 7-ton derrick that also unloaded it from the barges. The cement was brought to the mixer by a belt conveyor from a cement shed on the wharf. After it was mixed the concrete was shot direct into bottom-dumping skips on wagons and hauled to the work by petrol locomotives on 24-in. Decauville track.

The compressed-air installation for the work which has been described consisted of two rotary compressors with a capacity of 1,020 cb ft. of air per minute at 15 lb per square inch and two delivering 610 cb ft. per minute at 15 lb per square inch. An emergency plant, comprising one of the larger com-

pressors coupled to a Diesel oil engine, was supplied for use in the event of a total failure of electric power. Ingersoll-Rand compressors supplied high-pressure air for tools. *Fig. 26* shows the arrangement of the compressed air distribution.

In addition to the work described, this contract included two cast-iron tunnels 7 ft. 3 in. internal diameter subsequently lined with concrete, connecting two shafts sunk in the river with those already referred to, sunk at the power house. Circulating water for the electrical machinery and for use in the factory is drawn from and returned to the river in these tunnels, and dredging has been carried out so that the tops of the river intake and discharge shafts are kept at a level at which they will always be submerged. The shafts are provided with screens. A dividing wall of No. 3 Larssen sheet piling 200 ft. in length separates these shafts to prevent the discharged warm water from entering the intake tunnel, and both orifices are protected from shipping by a timber staging.

The works comprised in this contract were commenced in August 1930 and substantially completed in September 1932.

The author desires to express his thanks for the permission to write these articles given on behalf of the Ford Motor Co., Ltd., by Mr. J. H. Boyd, M.I.A.E., Chief Engineer of the Power and Construction Department, who was engineer for the whole of the works at Dagenham, and to Sir Cyril Kirkpatrick, PP.Inst.C.E., under whose direction as consulting engineer the works described in this paper were designed and constructed, and for whom during the latter half of the work the author acted as resident engineer. The main contractors for the works described were Messrs. John Mowlem & Co., Ltd.

(Concluded.)

Book Reviews.

"Mittig gedrückte Säulen. By A. Kleinlogel and K. Hajnal-Kóni.

Berlin: W. Ernst & Sohn. Price: R.M. 4 60.

THIS is the second part of the authors' book dealing with examples of calculations in reinforced concrete design, and covers the various methods adopted in detailing axially-loaded columns with hooped or spiral binding. A section is also given treating the modifications required when the slenderness ratio of the column is greater than 15 for hooped columns or 13 for columns with spiral wrapping. The examples included are worked in detail.

"Behälter, Maste, Schornsteine, Rohrleitungen," ("Handbuch für Eisenbetonbau," Vol. IX), Parts 1 and 2. By Dr. F. Emperger.

Berlin: W. Ernst & Sohn. Price: R.M. 5.50 each.

THE first two parts of this volume, dealing with the design and construction of reservoir, poles, chimneys and pipes, have

been written by Professor B. Loser, of Dresden, and carry the subject matter up to special classes of tanks required in certain industries.

The opening chapter deals with the materials used in making concrete tanks and the means adopted for securing impermeability. Considerable attention is given to special waterproofing and acid-resisting compounds. This is followed by a list of completed tanks and their dimensions and wall thicknesses. To English readers the drawings of details of reservoirs and tanks will prove useful, as indicating the principal features of German design. A large number of completed structures is shown, including reservoirs below ground, elevated water tanks, gasholder tanks, and swimming baths. Many of these are of unusual dimensions and recent construction.

Reinforced Concrete Residential Flats.

At Cornwall Gardens, Kensington, S.W., a block of flats has recently been completed to the design of the Estate Department of the Prudential Assurance Co. This building is noteworthy as being the first of its kind erected in London in which the whole of the structural work is executed in reinforced concrete. The elevations (*Fig. 1*) are in white Portland cement concrete. At fourth floor level

block. The planning of the first, second and third floors is similar, with four flats on each floor. At fourth-floor level a wide verandah has been built on the north end of the block, and a narrower verandah on the south and east sides.

Construction.

The foundations were taken down to the London clay, and consist of isolated

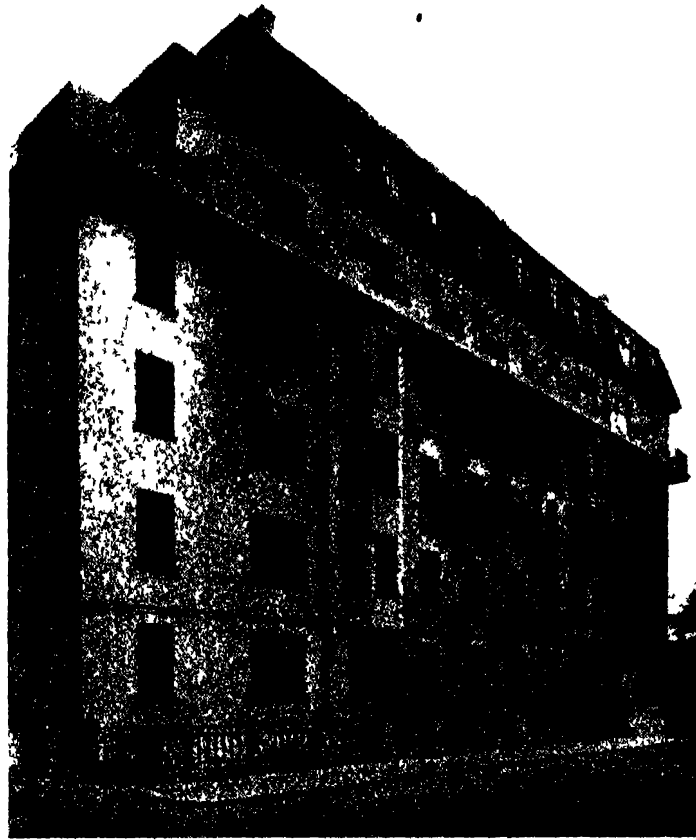


Fig. 1.

there is a balcony with reconstructed stone corbels and tympanum.

There are twenty-four residential flats divided into three types according to the accommodation provided. In the basement are the coal and coke stores, luggage store, electricity sub-station room and meter room, store rooms, porters' room, and a flat for the resident porter. Access to the basement is by stairs and lifts.

At ground-floor level is the main entrance hall which extends as a central hall from front to back. Four flats are provided on this floor. Each flat is situated at a corner of the rectangular

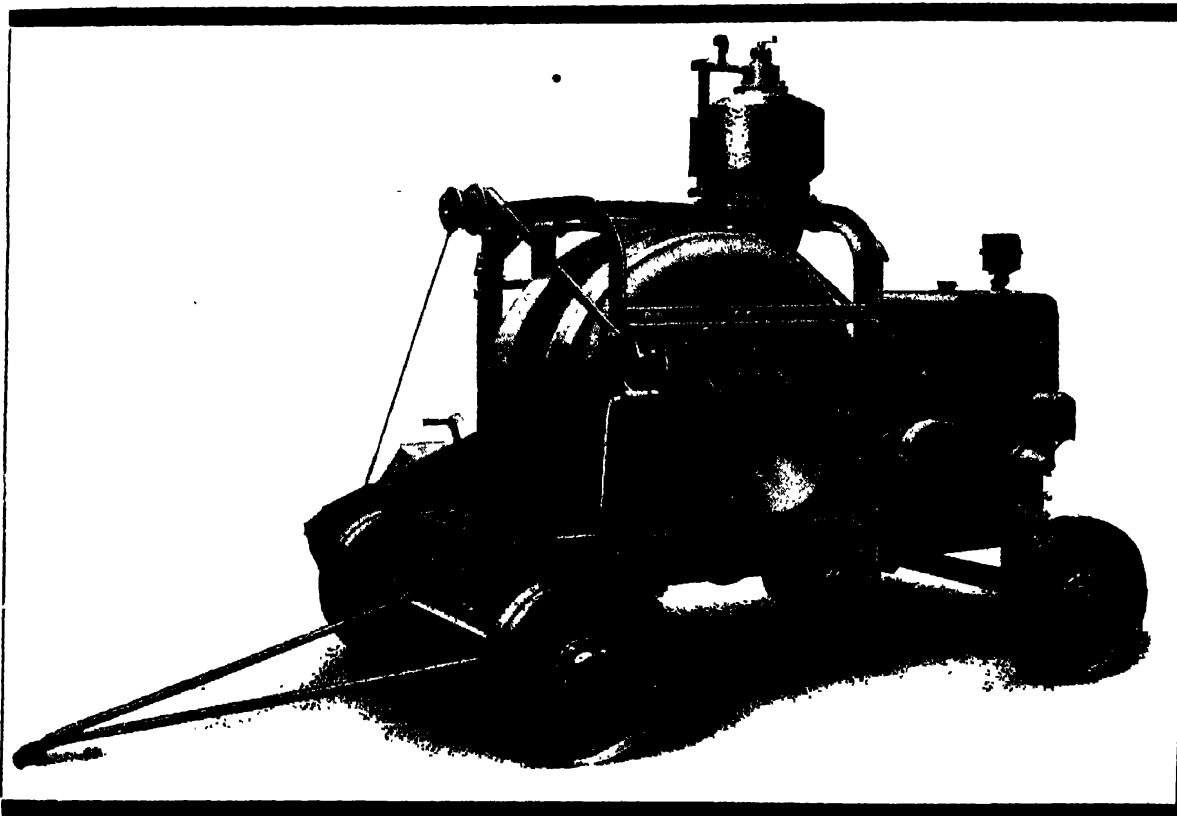
reinforced concrete footings. These are square in plan, their dimensions being proportional to the column loads. The reinforcement consists of a mat of $\frac{5}{8}$ -in. bars at 6-in. or 7-in. centres. The columns are square in cross section, the largest having a 27-in. side and being reinforced with twelve $1\frac{1}{4}$ -in. vertical bars and links. This column stands on a pyramid-shape footing 11 ft. by 10 ft. 3 in. in plan having a 9-in. reinforced slab at the bottom. In the columns and footings the proportions of the concrete are 1 : 2 : 4.

Below the basement floor*are special

October, 1933.



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Batcherplants

ON WALLASEY SEA WALL

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“Blawforms”

“Blawforms,” which have been used on over 31,000 contracts, were also used on the Wallasey Promenade Improvement Scheme, because of their simplicity and economy for shuttering work for concrete structures.

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floor beams continuous between two columns and with cantilever extensions to carry the party wall without bringing the exterior row of column footings close to the line separating the two buildings. In a similar manner cantilever beams were inserted in the basement to carry a portion of the building over the Metropolitan Railway tunnel at the junction of Stanford Road and Cornwall Gardens

was adopted the design was altered to take advantage of the reduced loading (40 lb per square foot) permitted under the code. The floors were constructed in concrete mixed in the proportions of 1 1½ 3. The construction consists of main tee beams of reinforced concrete between which the hollow tile floors span. Typical main beams are 11 in deep overall and 7 in wide in the web with

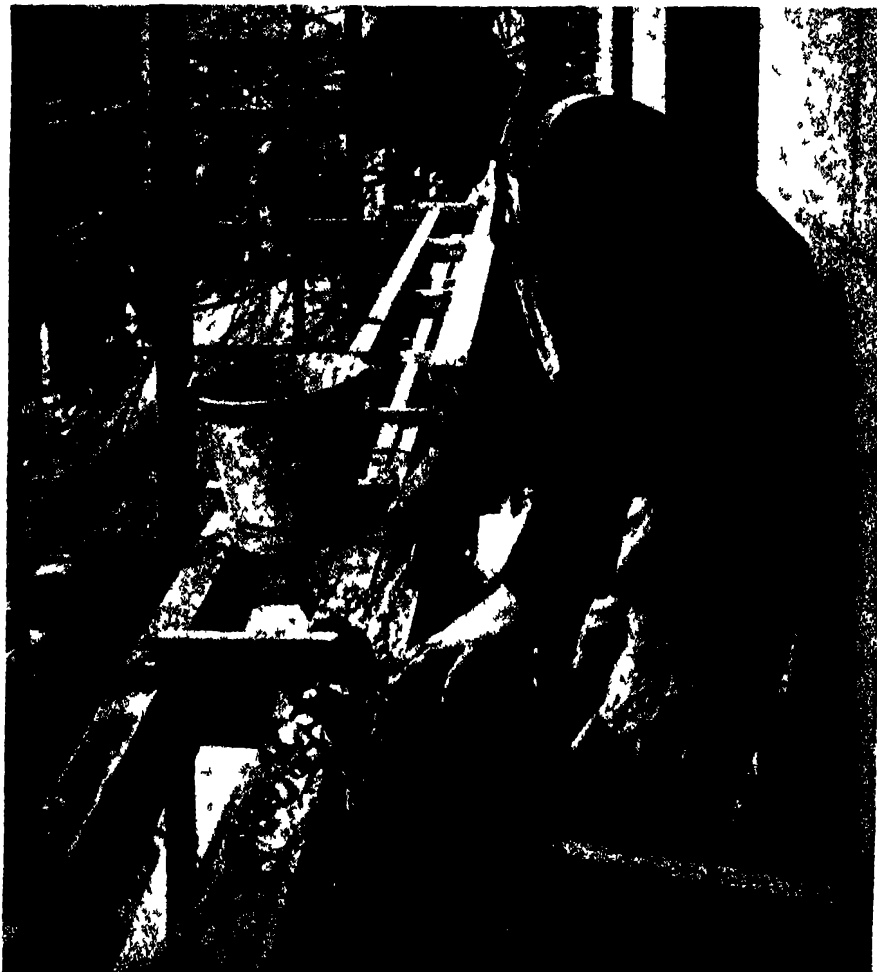


Fig. 2.

When the original drawings were made it was not expected that this tunnel would be met with in the excavation consequently a re-arrangement in the design of the basement became necessary and the difficulties of construction were considerably increased during the period from June to November of last year.

Originally the floors were designed to carry a superimposed load of 70 lb per square foot as required by the L.C.C. Regulations, but after the "Steel Code"

flanges 17 in wide, the cross sections of the beams varying with the spans. Between these beams the hollow-tiled floor consists of 10 in by 4½ in tiles at 13-in centres, giving a 3 in space to form the web of the secondary reinforced concrete beams. The flanges of the latter are 2 in deep, the tops of the tiles being set at this distance below the floor level, and are reinforced by ½-in bars at 12-in centres. All suspended floors were screeded for wood-block surfacing.

The stairs are constructed of reinforced concrete cast *in situ*, and finished in granolithic on which carpeting has been laid.

Walling.

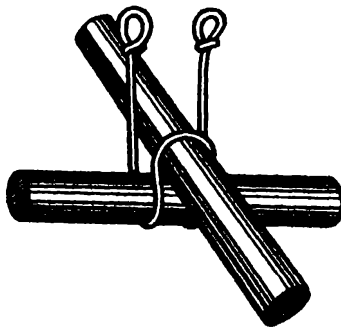
There are two classes of interior walling, (1) a 2½-in. hollow tile wall plastered on both sides for use as internal partitions, and (2) for the partition walls between pairs of flats a wall consisting of two skins of 2½-in. hollow tiles with a 2-in. air space between, and 2½-in. molar solid blocks supplied by Messrs. Sankey for the staircase partitions. Partitions are constructed directly on the floor as required.

The exterior walling consists of a 4½-in. reinforced concrete slab, a 2-in. air space and a 2½-in. inside tile with plastering; the materials on the inner side of the 4½ in. reinforced concrete slab are required to conform to the L.C.C. requirements for fire-resisting construction, the slab itself being capable of resisting any possible combination of structural and wind loading. In the reinforced concrete slabs

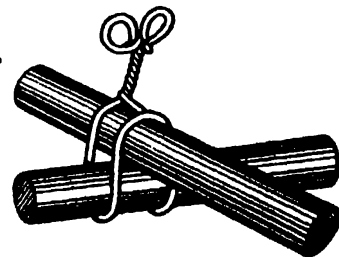
the reinforcement consists of a mesh of ⅝-in. horizontal bars at 12-in. centres and ¼-in. vertical bars at the same centres.

Fig. 2 illustrates the construction of a portion of the walling on the balcony at the fourth floor level. A sheet metal moving shutter was used to separate the white concrete facing from the grey concrete backing. The front and back forms for the wall consisted of ⅝-in. waterproof plywood built into panels with stiffened edges. These proved very successful and some were used as many as fifty times before they were scrapped. The thickness of the ordinary concrete is 4 in. and the white Portland cement concrete facing is 1 in. thick. The steel plates used were ⅛-in. thick, 6 ft. to 7 ft. long, and 18 in. high. Both the facing and backing concrete were placed in 18-in. lifts before the men raised the steel plates to allow them to bond together.

The contractors were Messrs. Hall, Beddall & Co., Ltd., and Messrs. F. Bradford & Co., Ltd., designed and constructed the reinforced concrete work. The steel reinforcement was supplied by the United Steel Co., Ltd.



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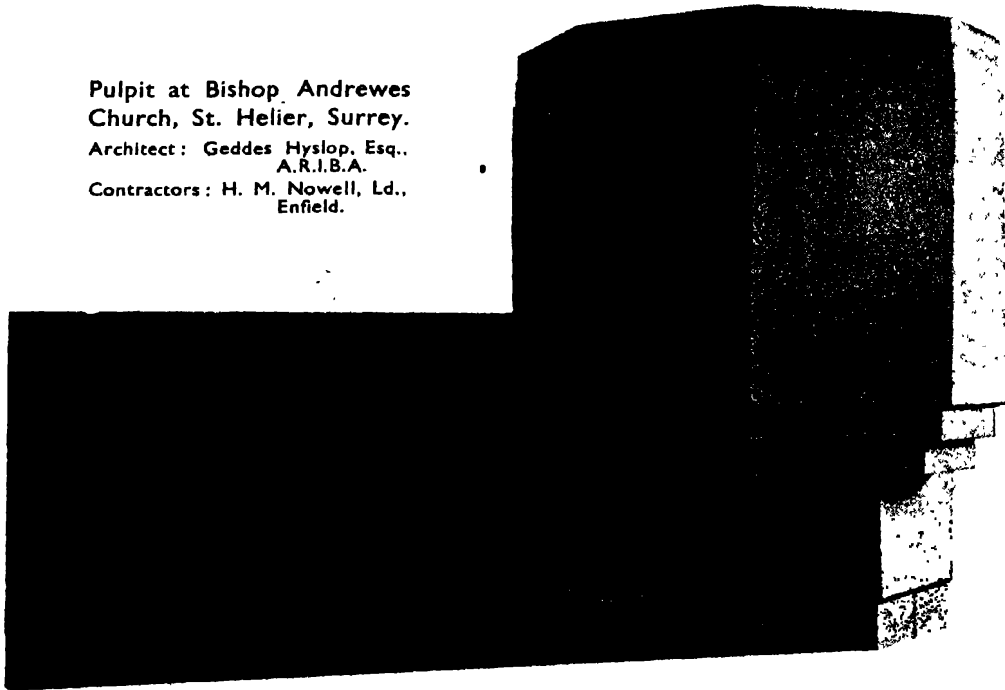
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October, 1933



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Architect: Geddes Hyslop, Esq.,
A.R.I.B.A.
Contractors: H. M. Nowell, Ltd.,
Enfield.



Example of concrete construction *in situ* with
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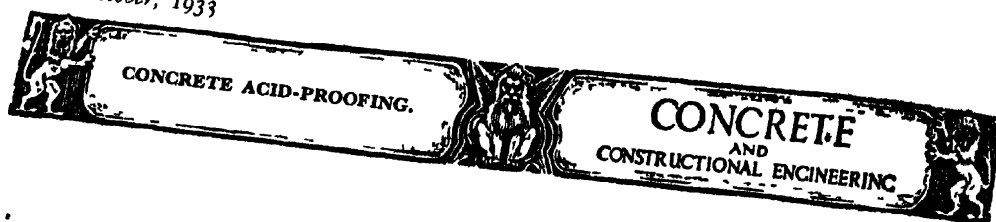
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October, 1933



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Recent Patents Relating to Concrete.

Structures of Reinforced Concrete.

376,259.—J H de W Waller, 115, Grafton Street, Dublin March 28, 1931

A method of building with reinforced concrete, cement, plaster, or like material comprises stretching woven flexible material composed solely of vegetable fibres, substantially in a straight line between supporting means to give it an initial tension and then applying one or more layers of concrete cement plaster, or the like to the flexible material which forms

to the pillars (5) and when stretched in place is covered with plaster (15) which is pressed through the meshes of the hessian to form a coating (16) The pillars (5) may be of timber

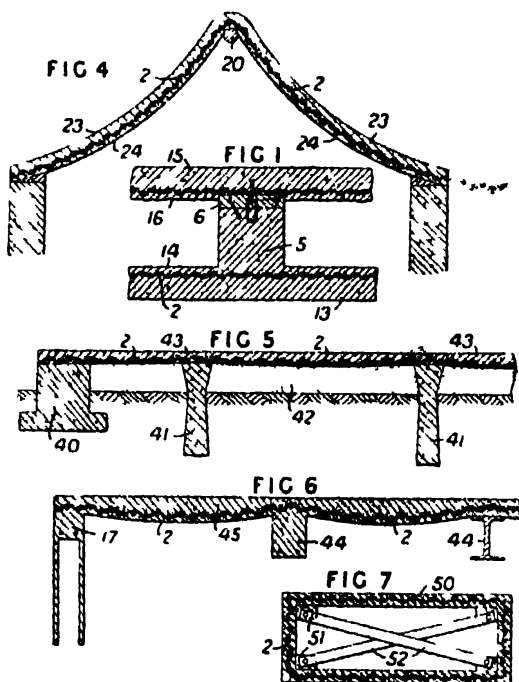
In the roof shown in *Fig 4*, hessian (2) is maintained in a stretched state over a ridge pole (20) Wire netting or other metal reinforcement may be placed under the hessian to prevent it sagging A layer of cement or concrete plaster (23) is placed over the outside of the hessian, and when this layer has set a layer (24) is placed on the inside, covering both the hessian and the wire netting

The application of the invention to floors is indicated in *Figs 5* and *6* In the construction shown in *Fig 5*, the floor is adapted to be supported by footings (40) and concrete supports (41) A layer of sand (42) is spread over the ground so as to be level with the supports and the hessian (2) is laid on top and when stretched is secured to the footings (40) Concrete (43) is then applied to the hessian to form the floor surface

An upper floor may be made as shown in *Fig 6* the hessian (2), which is stretched over the floor joists (44) and fixed to the top of the wall plate (17), being covered with concrete (45)

Floors or walls may be built from units, such as shown in *Fig 7*, comprising hollow tubular members (50) reinforced with hessian (2) The members are built around a framework of parallel members (51), preferably of timber, which are held in place by readily removable bracing (52)

To form a floor, the members (50) are laid side by side the joints are grouted, and the whole covered with concrete, the undersides being, if desired, plastered to form a ceiling A space may be left between any two tubular members, and a reinforcing bar placed in this space which is filled with concrete to form a joist To form a wall, the tubular members are laid horizontally or vertically and bonded together by cement mortar, the outside and inside being covered with a thin layer of concrete, cement, plaster, or like material The invention may be applied to the construction of beams and stairs



STRUCTURES OF REINFORCED CONCRETE

the reinforcing means and is kept under permanent tensile stress in the finished structure In the double wall shown in horizontal section in *Fig 1* hessian (2) for the outer wall is secured at one end to a rail (not shown) supported by concrete pillars (5) and after being stretched is fastened to one or more other supports according to the length of the wall The hessian is preferably wetted to shrink it and is then covered with a thin paste of concrete or cement, and when dry is covered with layers (13, 14) to form the outside wall The hessian for the inner wall is fastened to timber strips (6) secured

Obituary.—We regret to announce the death of Mr Cecil Sainsbury who was with Messrs Stothert & Pitt, Ltd, for 43 years, and for many years joint managing director of the Company.

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Clean Shingle, $\frac{3}{4}$ in. mesh	"	6	9
" " $\frac{1}{2}$ in. mesh	"	9	0
Thames ballast	"	6	9
Broken brick ($\frac{3}{4}$ in.)	"	10	6
Best British Portland Cement (delivered London area) per ton 46s., including non-returnable paper bags; 44s. 9d., including charge for hire of jute sacks.			
Rapid-Hardening Portland Cement	delivered London	7s. 6d.	per ton extra.
"Colorcrete" red and buff rapid-hardening Portland Cement, delivered London	20s.		per ton extra
Coloured cement and aggregate mixtures	delivered London area	£5 to £5 10s.	per ton.
"Snowcrete" White Portland Cement, £8 5s. per ton delivered London, including non-returnable paper bags.			
"Super-Cement" Waterproof Cement	(including paper bags) per ton	76s.	
BOARDING FOR SHUTTERING—			
		Sawn.	Wrot.
1 in.	per square	22s. 6d.	26s. 6d.
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1¾ in.	"	34s. 0d.	40s. 0d.
SAWN TIMBER FOR STRUTS AND SUPPORTS—			
3 in. by 4 in.	from	£19	per standard
3 in. by 6 in. and 3 in. by 7 in.	"	£18	"
MILD STEEL RODS FOR REINFORCEMENT—			
			s. d.
$\frac{5}{16}$ in. to 2½ in. Rounds	per cwt.	8	6
$\frac{3}{8}$ in. to 1 in. Rounds	"	8	9
$\frac{1}{2}$ in. Rounds	"	9	0
$\frac{5}{8}$ in. Rounds	"	10	0
Breeze Slabs per yd. super: 2 in., 1/6; 2½ in., 1/8; 3 in., 2/-; 4 in., 2¼.			

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(Based on Contracts up to £2,000.)

PORTLAND CEMENT CONCRETE WELL MIXED IN PROPORTION 1 : 2 : 4—		s.	d.
Do. do. in foundations	per foot cube	1	3
Do. do. in columns	" "	1	5
Do. do. in beams	" "	1	7
Do. do. in floor slabs 4 in. thick	per yard super	3	8
Do. do. in floor slabs 5 in. thick	" "	4	7
Do. do. in floor slabs 6 in. thick	" "	5	6
Do. do. in floor slabs 7 in. thick	" "	6	5
Do. do. in walls 6 in. thick	" "	5	6

(Add for hoisting 3s. 6d. per yard cube above ground-floor level. Add for rapid hardening Portland Cement 2s. per yard cube.)

STEEL REINFORCEMENT, INCLUDING CUTTING, BENDING AND PUTTING INTO**POSITION AND SECURING WITH STOUT BINDING WIRE—**

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From $\frac{1}{4}$ in. to $\frac{3}{8}$ in.	per cwt.	16 0
" $\frac{7}{16}$ in. to $\frac{1}{2}$ in.	"	15 0
" $\frac{9}{16}$ in. to 2½ in.	"	14 0
EXTRA LABOUR TO BENDS in ¼-in. rods, ½d.; ⅜-in. rods, 1d.; ½-in. rods, 1½d.; ⅝-in. rods, 1½d.; ¾-in. rods, 1½d.; 7/8-in. rods, 2d.; 1-in. rods, 2½d.; 1½-in. rods, 3d.; 1½-in. rods, 3½d.; 1½-in. rods, 4½d.; 1½-in. rods, 6d.		
EXTRA LABOUR TO HOOK BENDS: ¼ in., 1d.; ⅜ in., 2d.; ½ in., 2½d.; ⅝ in., 3d.; ¾ in., 3½d.; 7/8 in., 4d.; 1 in., 4½d.; 1½ in., 6d.; 1½ in., 7d.		

SHUTTERING—

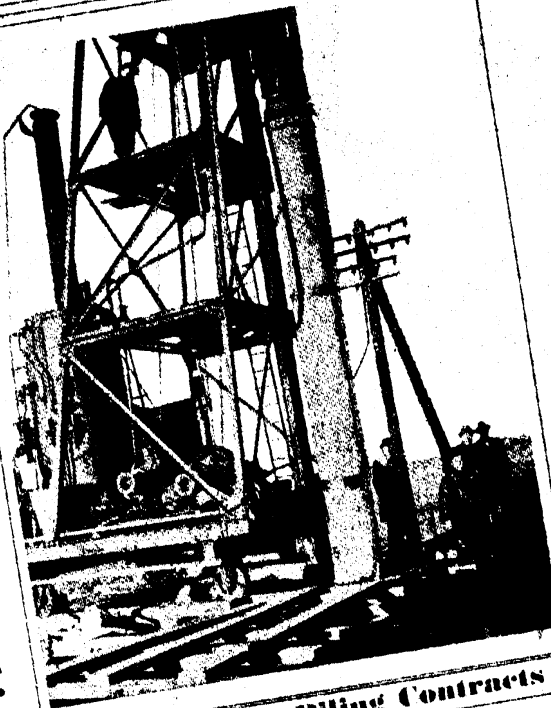
	s.	d.
Shuttering and Supports for Concrete Walls (both sides measured) per square	50	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	46 0
Do. do. in small quantities	per ft. super	0 8
Shuttering and Supports to Stanchions, average 18 in. by 18 in.		
	per ft. super	0 7½
Do. do. as last, in narrow widths	" "	0 9½
Do. do. to sides and soffits of beams, average 9 in. by 12 in.	" "	0 9½
Do. do. as last, in narrow widths	" "	0 10
Raking, cutting, and waste to shuttering	per ft. run	0 3
Labour, splay on ditto	" "	0 2
Small angle fillets fixed to internal angles of shuttering to form chamfer	" "	0 3

WAGES.—The rates of wages on which the above prices are based are:—Carpenters and joiners, 1/7 per hour; Carpenters working on old shuttering, 1/8; Labourers on building works, 1/2½; Men on mixers and hoists, 1/3½; Bar-benders, 1/3½.

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Prospective New Concrete Work.

ABERDEEN —Quay Wall —The harbour authorities have approved a scheme for reconstructing the quay wall at an estimated cost of £8,000

ALLHALLOWS —Concrete Pool —The Hoo RDC has approved a proposal to construct a concrete swimming pool at Allhallows

ARBROATH —Bathing Pool —The IC has decided to construct a sea water bathing pool at Seaforth House at an estimated cost of £8,000

BLDWORTH —Swimming Baths —The UDC proposes to construct open air swimming baths at the Heath Road recreation ground

BLISTON —Flood Prevention —The Bliston UC proposes to construct flood prevention works at Pasture Dyke at an estimated cost of £17,000

BICKERTON —Water Supply —The Nantwich RDC proposes to construct water supply works for Bickerton at an estimated cost of £12,200

BIRMINGHAM —Flood Prevention —The IC has applied for a loan of £9,500 for flood prevention works at Park Hill, Harborne.

BODMIN —Swimming Pool —The TC is considering the construction of a swimming pool

BOOTLE —Sewerage —The TC has approved a drainage scheme for the Rimrose Brook district at an estimated cost of £282,000

BOSTON —Bridge —The TC is considering the widening of Bargate Bridge, on the Boston-Skegness road

BRLNTWOOD —Sewage Disposal —The UDC has received sanction to borrow £26,447 for the extension of the Nag's Head Lane sewage disposal works

BURNTISLAND —Swimming Bath —The TC is considering a proposal to construct a swimming bath at an estimated cost of £8,000

CARDIFF —Weir —The TC proposes to construct a weir in the river Taff

CONGLETON —Swimming Pond —The TC is considering proposals for constructing an open-air swimming pond

CORBY —Water Tower —The Kettering RDC is to construct a water tower near Rockingham Road with a capacity of 123,000 gallons

CORNFORTH —Flood Prevention —The

PC proposes to carry out flood prevention works at West Cornforth

CROMER —Groyne —The UDC is to construct a groyne at the east boundary of the foreshore at an estimated cost of £2,250

DOVER —Road —The TC is considering a proposal to widen Archcliffe Road at an estimated cost of £8,000

DUNDLE —Swimming Pool —The BC is proposing to construct a swimming pool on the beach at an estimated cost of £35,000

DUN LAOGHAIRE —Swimming Pond —The TC is considering the construction of an open air swimming pond

DUNOON —Bathing Pool —The IC is to proceed with the construction of a bathing pool at Moir Street

GREAT IDSTONE —Reservoir —The Kirbymoorside RDC proposes to construct a reservoir at Great Idstone

GRINDLETON —Bridge —The Bowland RDC has approved the construction of a bridge and approaches at Grindleton, at an estimated cost of £5,000

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HILLSIA — *Sea Wall, etc* — The Portsmouth T C has applied for sanction to borrow £10 650 for a sea wall and £22 670 for the construction of a swimming bath at Hillsca

LAMBOURN — *Water Supply* — The Hungertord R D C is proposing to undertake a water supply scheme for the Lambourn district at an estimated cost of £21 763

LLANIDRY — *Swimming Pool* — The T C is considering the construction of a swimming pool at the Lake Lands Parc Howard

LONDON (WOOD GREEN) — *Swimming Pool* — The U D C has applied for sanction to a loan of £26 194 for the construction of an open air swimming pool at Durnsford Road

LONG FATON — *Sewage Disposal* — The U D C has applied for a loan of £42 750 for sewerage and sewage disposal works at Stapleford

LUTON — *Sewage Disposal* — The T C has applied for sanction to borrow £4 000 for sewage disposal works

MANBY — *Aerodrome* — The Air Ministry is to construct an aerodrome at Manby near Grimsby

MARCHION — *Water Supply* — The Gwyrfa R D C has approved a water supply scheme for Llunfairisgar, Llunwig and Llanddennollen at an estimated cost of £15 000

MARGATE — *Reservoir* — The T C proposes to erect a reservoir with a capacity of 5 000 000 gallons

MARGATE — *Sea Wall* — The T C proposes to complete the sea wall and promenade at Foreness Bay

MERTHYR — *Bridge* — The T C is to reconstruct Brandy Bridge

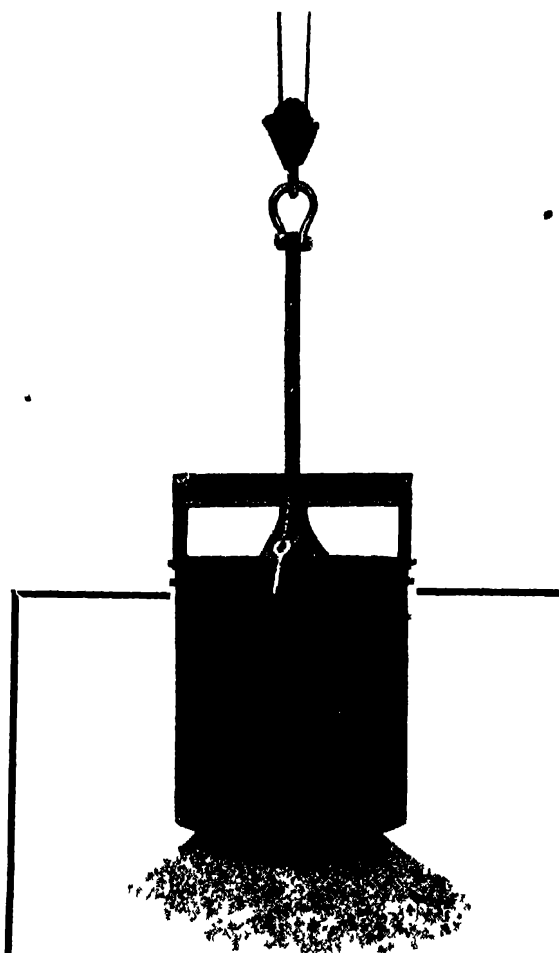
MINIHAD — *Swimming Bath* — The U D C is to proceed with the construction of a swimming bath

MOUNTAIN ASH — *Swimming Bath* — The U D C proposes to construct a swimming bath

NANTWICH — *Open air Bath* — The U D C has approved the proposed construction of an open air swimming bath on the Salt Ley Meadow, at an approximate cost of £4 000

NANTWICH — *Water Supply* — The R D C is considering a scheme for the construction of a reservoir, pumping station and ancillary works in connection with a water supply for Bickerton at an estimated cost of £12 200

NEWBIGGIN — *Breakwater* — The U D C is considering the construction of a breakwater



THE "WINNER" SKIP

(Patent pending No 13314/33)

**DISCHARGES CONCRETE
FROM THE BOTTOM &
ELIMINATES TIPPING**

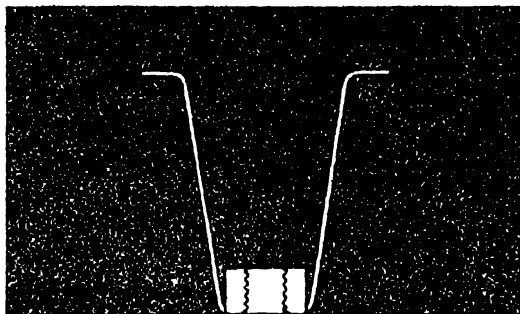
The "WINNER" SKIP (Patent pending No 13314/33) discharges concrete in a uniform mixture without segregation of materials from the bottom, and eliminates tipping. No alterations are required to the crane, and no chains or "dead lines" to adjust by the Crane driver. Hang the skip on in the ordinary way, and place the cotter in the spindle at the loading station, and withdraw it at the discharging point.

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RAMSGATE — *Water Supply* — The T C has applied for sanction to borrow £12,500 for the extension of the water adits

RICHMOND (SURREY) — *Swimming Bath* — The T C is considering a scheme for the construction of an open-air swimming bath

SALFORD — *Bridge* — The T C has decided to reconstruct Cromwell Bridge, connecting Pendleton and Broughton, at an estimated cost of £30,000

SHOREHAM BY-SEA — *Bathing Pool* — The U D C proposes to construct an open-air swimming pool at the Adur Memorial Recreation Ground

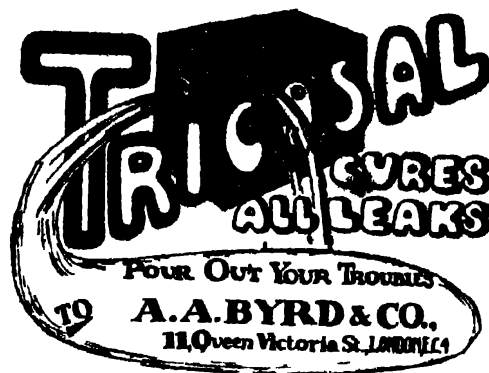
SHREWSBURY — *New Road etc* — It is stated that the T C has decided to construct a new loop road from Welsh Bridge to Shoplatch, and to extend the car park at Barker Street to allow for an omnibus station

SOUTHAMPTON — *Sewage Disposal* — The T C has applied for sanction to a loan of £236,000 for sewerage and sewage disposal works

STONEHAVEN — *Swimming Pool* — The T C is to construct a swimming pool at an estimated cost of £5,500

SUNDFRLAND — *Roads* — The T C has received sanction to a loan of £23,035 for road reconstruction work

TAMWORTH — *Sewage Disposal* — The T C and the R D C have applied for loans of £4,790 and £3,702 respectively for the construction of sewerage and sewage disposal works



THIRSK — *Water Supply* — The R D C is to apply for a loan of £4,720 for water supply works

TOLINS — *By pass Road* — The Devon C C has been recommended to approve the proposal of the Totnes I C to construct a by pass road, at an estimated cost of £20,000

WARRINGTON — *Sewage Disposal* — The R D C proposes to construct sewerage and sewage disposal works in the districts of Winwick, Poulton, Croft and Woolston at an estimated cost of £64,274. A further scheme is proposed to enlarge the sewage works at Burton Wood at an estimated cost of £7,847

WILLINGTON — *Road Improvements* — The U D C has under consideration a road improvement scheme near Burns Farm, at an estimated cost of £20,000

WOKING — *Bathing Pool* — The U D C is considering a proposal to construct a bathing pool in the Constitution Hill recreation ground at an estimated cost of £13,500

WORKINGTON — *Swimming Bath* — The T C has applied for a loan of £8,000 for the construction of a swimming bath

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Tenders Accepted.

CHICHESTER.—*Groynes*.—The M.C. has accepted the tender of A. G. Osenton, Hayling Island, for the construction of groynes at Chichester Harbour.

DONCASTER.—*Borehole*.—The C.B. has accepted the tender of the François Cementation Co., Ltd., Doncaster, at £1,474, for sinking a borehole in connection with the Thorham Farm (Cantley) water supply.

GLASGOW.—*School*.—The B.C. has accepted the tender of Wilson Bros., at £12,602, for reinforced concrete work in connection with St. Charles's School extension.

KINGSCLERE.—*Water Supply*.—The R.D.C. has accepted the tender of Reynolds & Son, Ltd., for the construction of a main and reservoir in connection with a water supply scheme for Kingsclere and Whitchurch.

LEICESTER.—*Concrete Floors, etc.*—The T.C. has accepted the tender of the Trussed Concrete Steel Co., Ltd., for reinforced concrete floors and staircases at the Corn Exchange.

LONDON (BARNES).—*Concrete Foundations*.—The tender of the Trussed Concrete Steel Co., Ltd., has been accepted for the reinforced concrete foundations for a block of maisonettes in Grove Road, Barnes.

LONDON, E.C.—*Bridges*.—The L.N.E. Railway Co. has accepted the tender of the Cleveland Bridge & Engineering Co., Ltd., Darlington, at approximately £24,000, for the reconstruction of three bridges near Fenchurch Street Station.

LONDON.—*Concrete Tank, etc.*—The tender of Walker-Weston Co., Ltd., has been accepted for the construction of a reinforced concrete tank of about 800,000 gallons capacity, and reinforced concrete foundations for three cast-iron tanks, in the London area.

SANDOWN (I. OF W.).—*Pier Reconstruction*.—The Sandown-Shanklin U.D.C. has accepted the tender of J. B. Edwards & Co. (Whyteleafe), Ltd., of London, at £22,774, for the demolition of a section of the promenade pier, and constructing in reinforced concrete the substructure of this section, and the erection of a concert pavilion.

SANDOWN (I. OF W.).—*Waterworks*.—The Sandown-Shanklin U.D.C. has ac-

cepted the tender of Aubrey Watson, Ltd., of London, at £8,891, for the construction of a sedimentation tank and ancillary work at Sandown waterworks.

SOUTHPORT.—*Waterworks*.—The C.B.C. has accepted the tender of F. Taylor & Co., of Littleborough, at £14,050, for the erection of a pump and filter house and ancillary works at Blundell House pumping station.

WEST BROMWICH.—*Sewage Disposal*.—The T.C. has accepted the tender of Tarmac, Ltd., of Ettingshall, Wolverhampton, for the construction of sewage disposal works at Ray Hall, the estimated cost being £60,000.

WHITBY.—*Concrete Pipes*.—The U.D.C. has accepted the tender of Fred Whitaker & Co., Ltd., Leeds, at £2,624 for laying about 150 yd. of 60-in., and 60 yd. of 39-in. diameter reinforced concrete pipes, manholes, etc.

Other tenders submitted: John Parkin, £2,640; John Pearson, Ltd., £2,797; F. M. Willers & Co., £2,890; Geo. H. Graham, £2,896; R. C. Crawford & Co., Ltd., £3,222; R. A. Wilson & Sons, £3,700; William Clark & Co., £4,050.

WOKING.—*Piles*.—The U.D.C. has accepted the tender of Simplex Concrete Piles, Ltd., at £220 for driving three piles in the site for the proposed swimming bath at Constitutional Hill.

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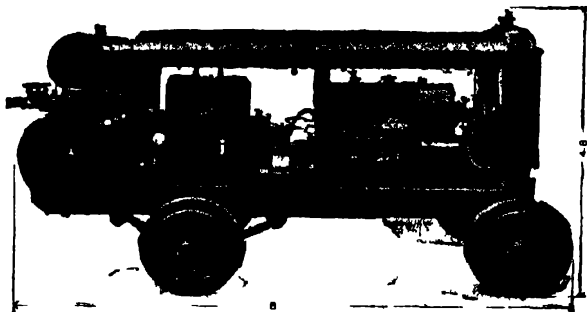
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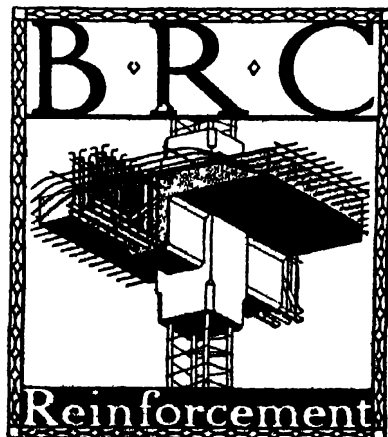


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The reinforced concrete balconies, shown in the photograph, have been adopted as standard construction in Liverpool. They are cantilevers and are cast at the same time as the Truscon reinforced concrete floors and wall lintels.

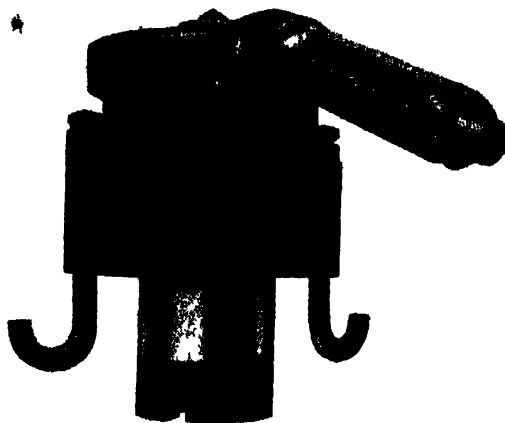
All the reinforced concrete work, comprising foundations, framing, floors and balconies, was detailed by The Trussed Concrete Steel Co., Ltd., whose knowledge and experience in reinforced concrete construction are readily available to all engaged in the important task of slum clearance and re-housing.

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Trade Notices.

Pneumatic Vibrators.—During the last two years increasing attention has been given to mechanical methods of vibrating the formwork or the reinforcement in concrete work so as to obtain a dense consolidated concrete. When the sections are thin and heavily reinforced hand tamping is not always satisfactory and it is difficult to avoid having air holes in the finished work. For such structures as chimneys, cooling towers, and tunnel linings mechanical vibration has proved useful.



PNEUMATIC VIBRATOR

and has been extensively employed. To overcome the need for bolting vibrators to the shuttering the Consolidated Pneumatic Tool Co., Ltd., has placed on the market an air clamp vibrator. When this is used rods on the sides of the formwork are arranged to suit hooks on the vibrator and when the throttle valve is opened the air pressure causes the hook bolts on the vibrator to grip the side rods, thus forcing the vibrator cylinder against the forms and setting up the vibrating action.

Surface Finishing Material.—A special cement is now supplied by the Cement Marketing Co., Ltd., which can be applied to concrete, brick, or stone when it is required to provide an inexpensive decorative treatment. This cement is known as 'Tintocrete' and forms a durable water resistant surface. For exterior work white and light cream colours are available while a range of bright colours may be obtained for interior decoration. In interior work 'Tintocrete' may be applied directly to walls, thus saving the cost of plastering. Instructions for using 'Tintocrete' are contained in a leaflet issued by the Cement Marketing Co. Either a paint brush or a high-pressure spraying machine may be used, the proportions of the mixture varying with the method of application.

Cast-in-situ Concrete Piles.—A brochure has been issued by the Franki Compressed Pile Co., Ltd., which contains a description of the method of driving Franki piles and their characteristic properties. Among the advantages to be gained by using the piles are stated to be (1) high carrying capacity—the normal working load is 80 to 110 tons on each pile, (2) driving to the exact depth without need for extending the pile or cutting off the head and (3) saving in length of piles due to the presence of the enlarged base. The piles can be driven on a batter and reinforced if necessary. A number of illustrations of work on which the Franki pile has been used is included, together with test results. In addition to the compressed pile the Company has developed a pile that can be sunk by hand and concreted under pressure, and the "Mega" pile which is jacked down in sections where underpinning is required.

Reinforced Concrete in Slum Clearance Schemes.—At the present time when plans are being prepared in many cities for the clearance of slums particular interest is attached to descriptions of the methods adopted in other countries. Practice on the Continent differs from that in Great Britain mainly in the erection of large blocks of tenements of modern design and centrally situated instead of separate small houses on building estates often far removed from the worker's original surroundings, and more important still at a considerable distance from his work. Some large cities now contain blocks of municipal flats where workers can live close to their work and in the surroundings to which they have always been accustomed. Reinforced concrete lends itself admirably to the construction of such buildings, as can be seen from the illustrations of modern blocks of flats contained in a booklet entitled "The Rôle of Reinforced Concrete in the Development of Replanned Slum Areas," issued by the British Portland Cement Association, which contains half-tone illustrations, drawings of work recently executed on the Continent and in Dublin, and a statement of the case for the erection of blocks of flats to replan slum areas. The pamphlet can be obtained free of charge from the Association.

Recent Patent Applications.

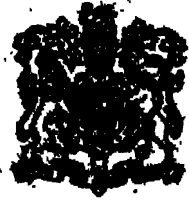
397,127. — M. N. Ridley : Reinforced concrete floors, roofs, and walls.
396,569. — F. Zollinger : Steel girders for concrete structures.

396,556. — G. A. Mannsell : Underpinning of bridges.

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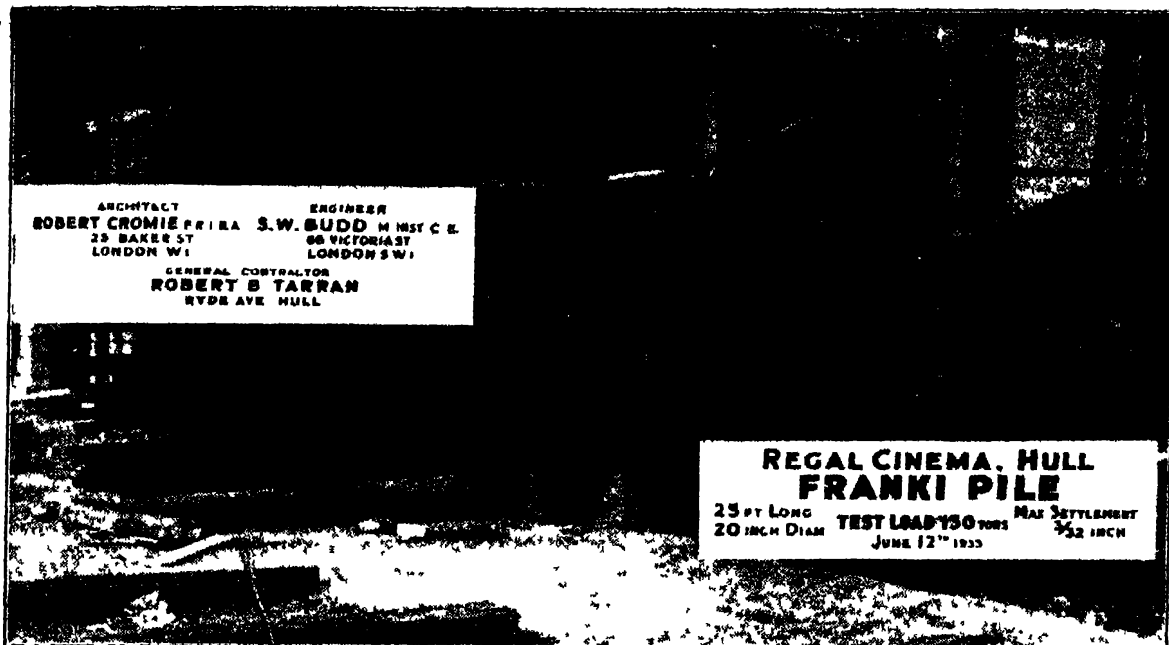


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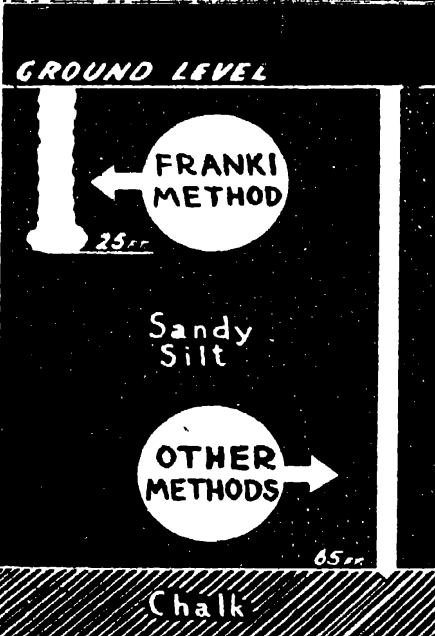


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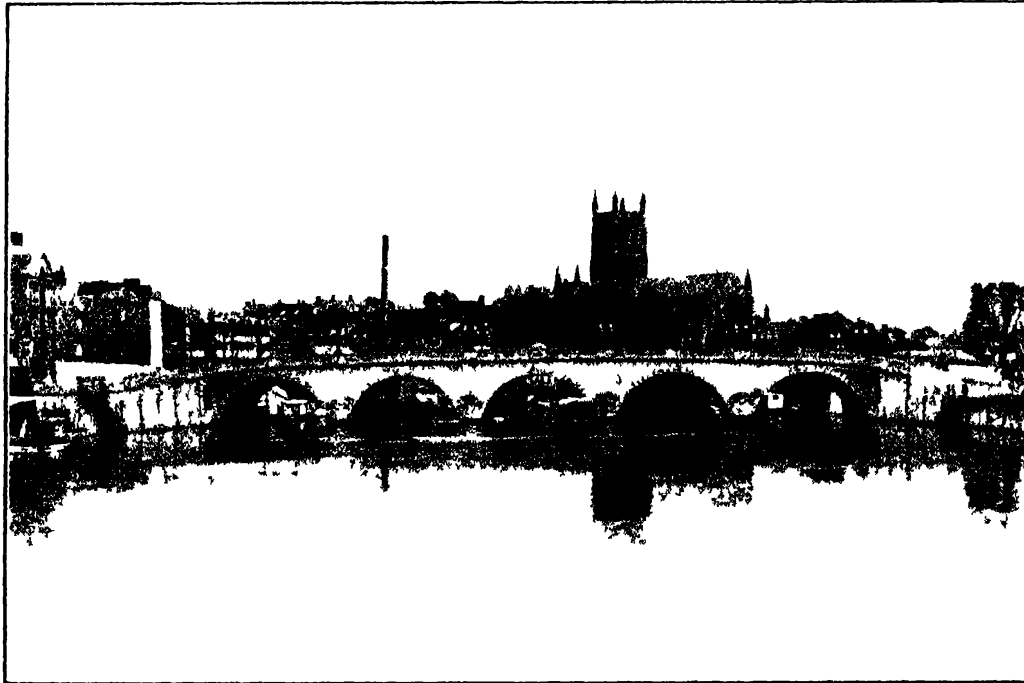


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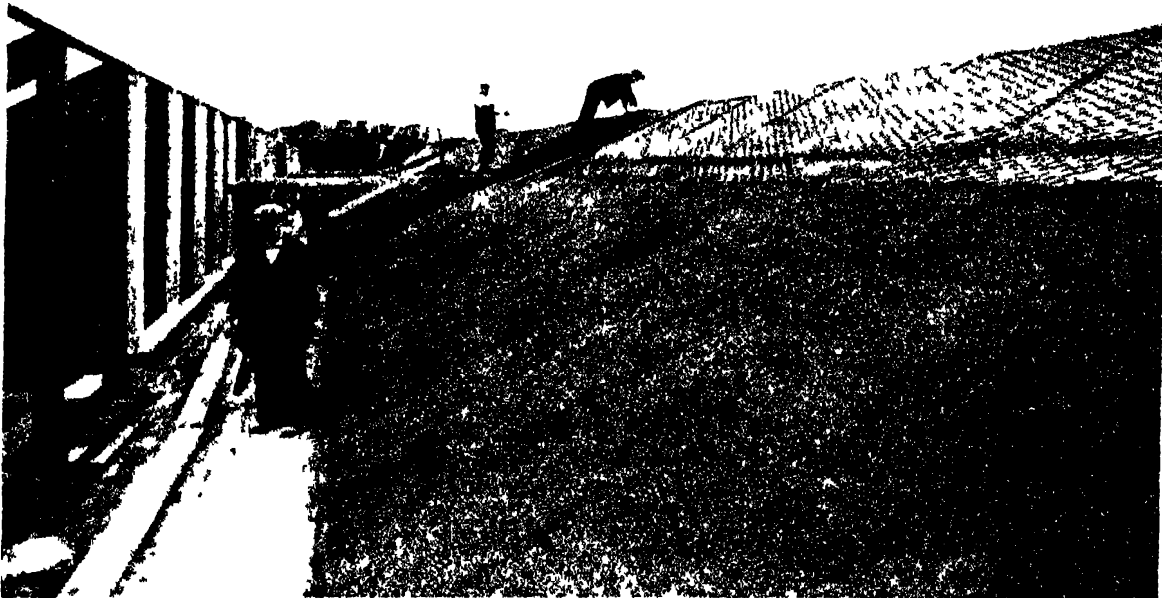
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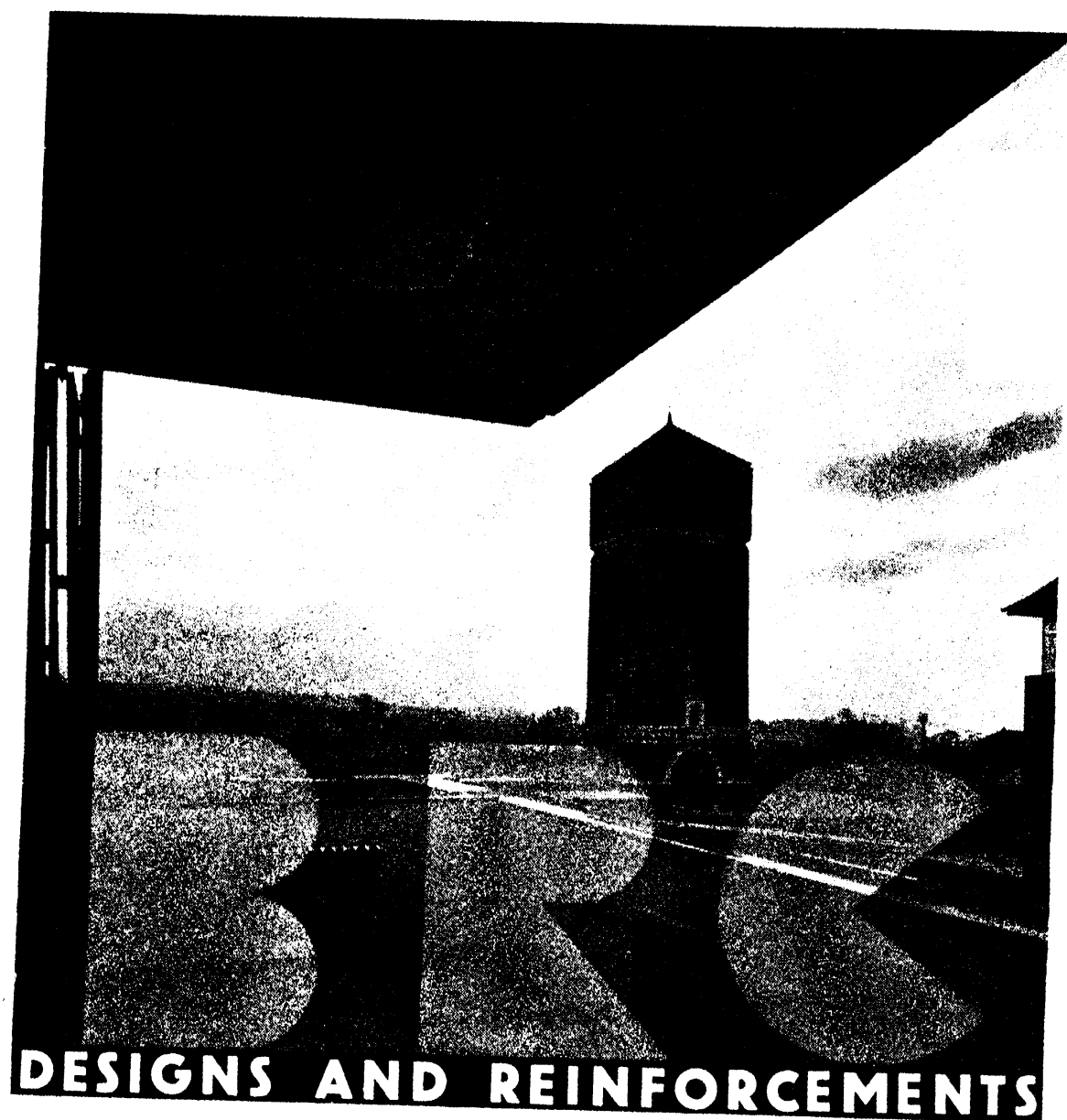
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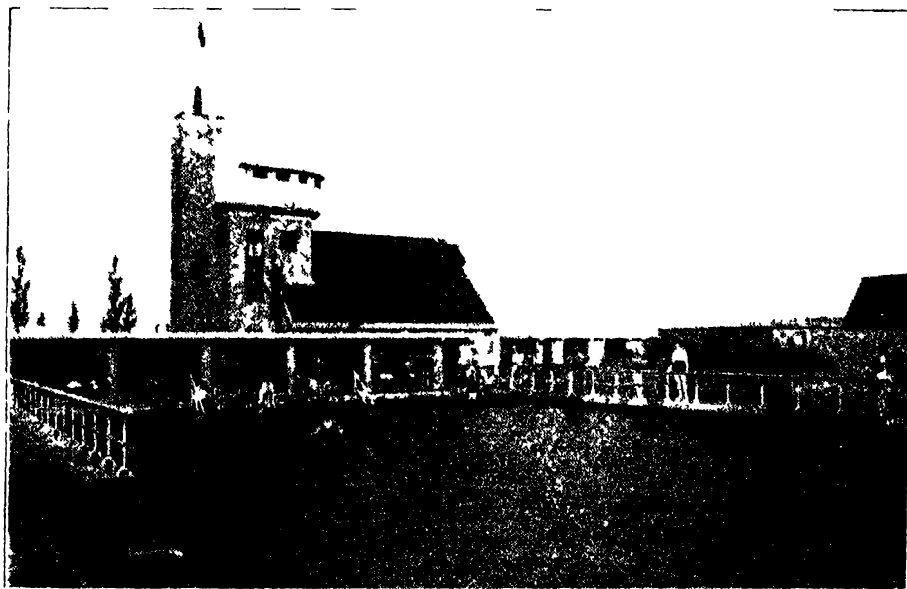
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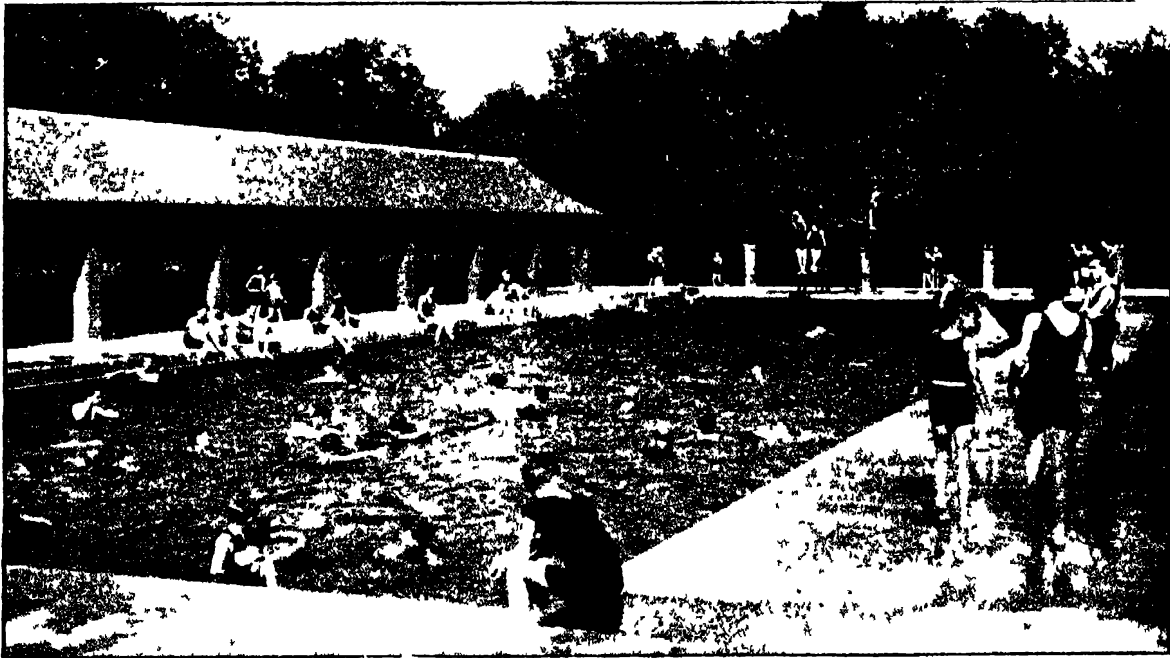
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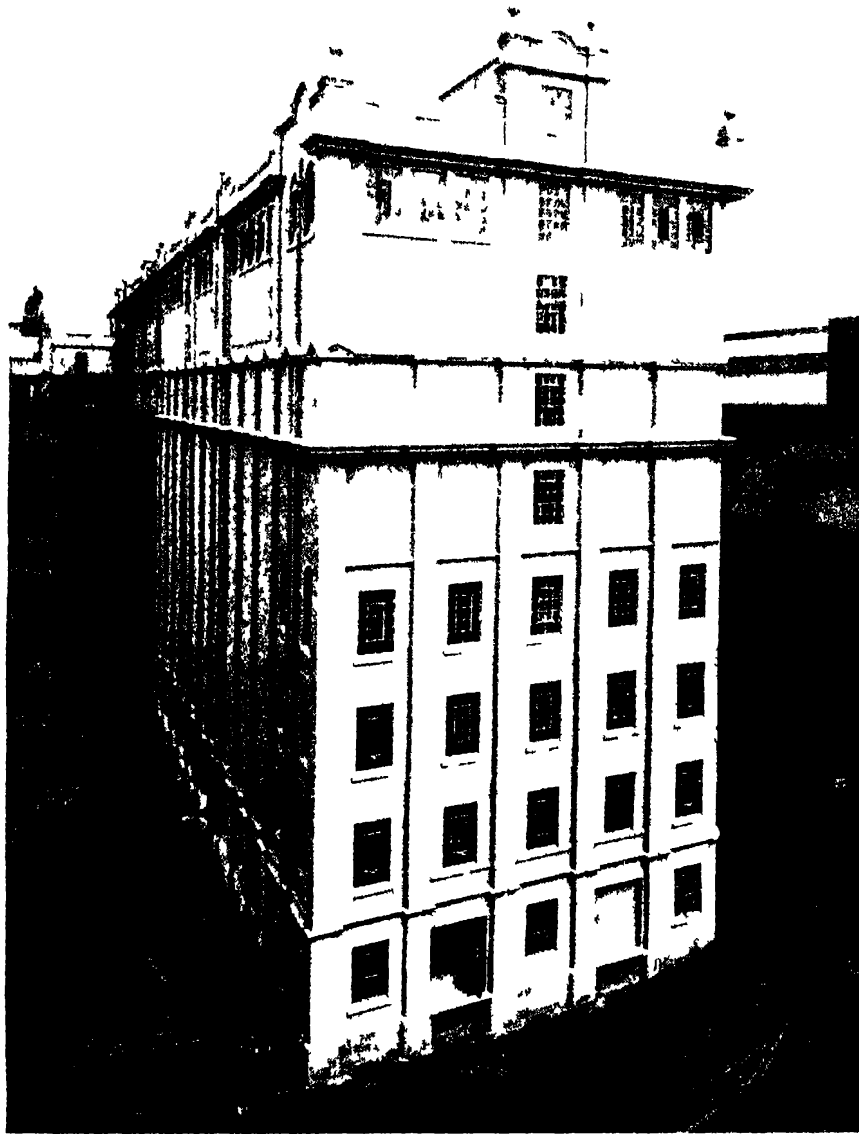
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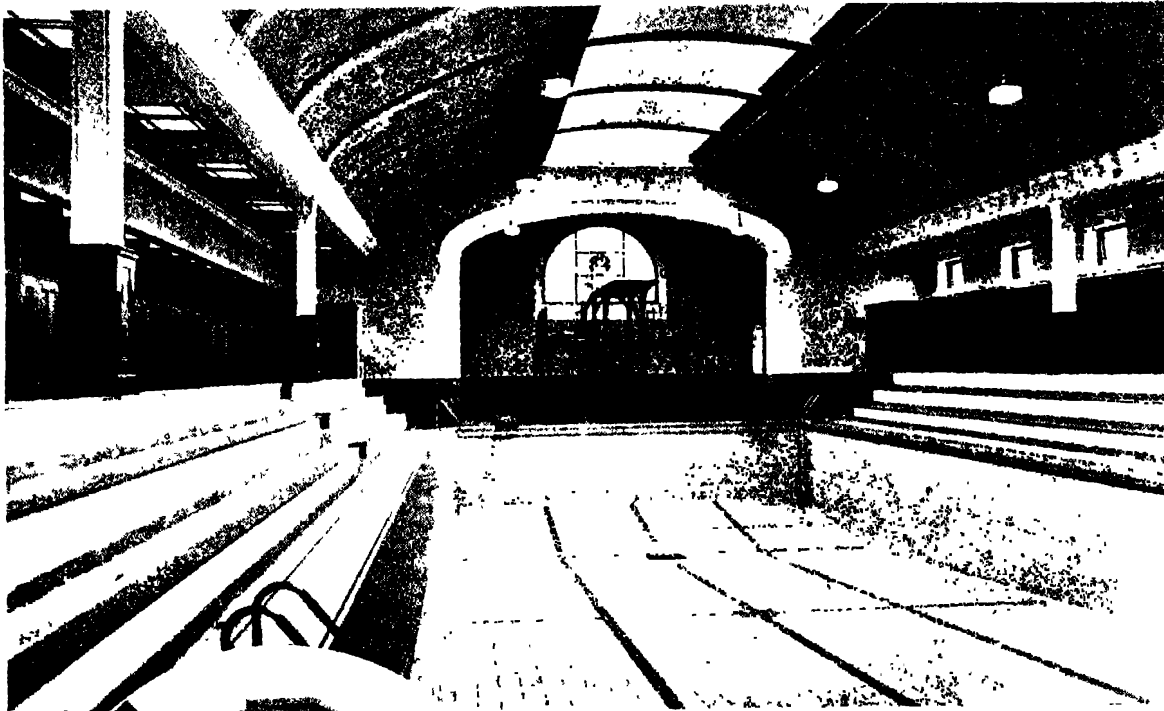
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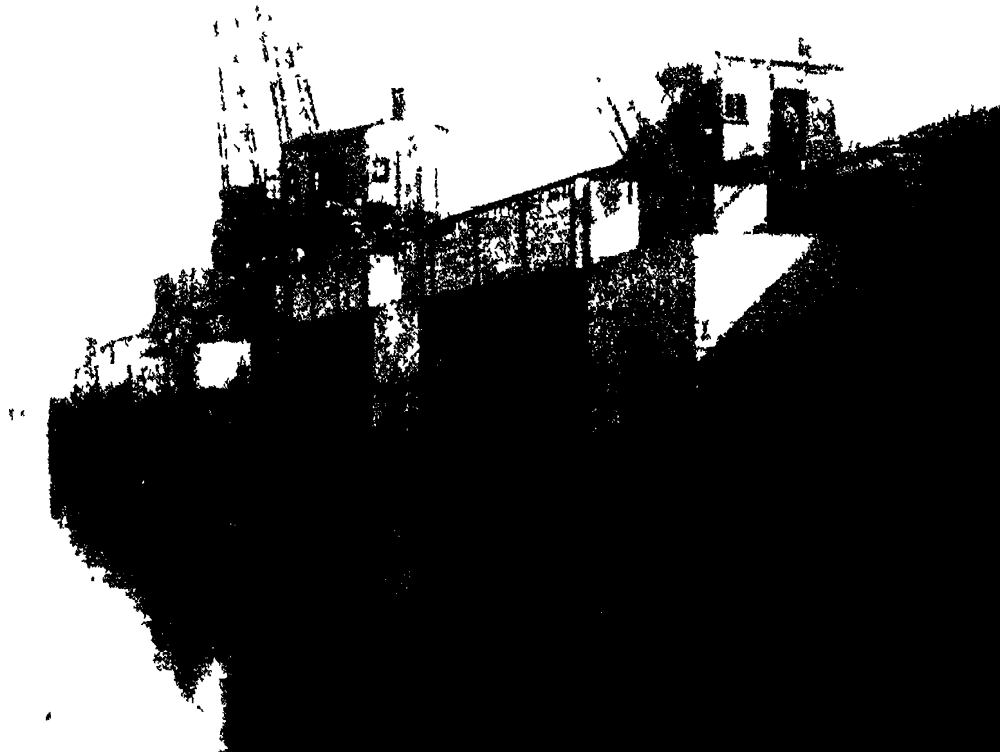
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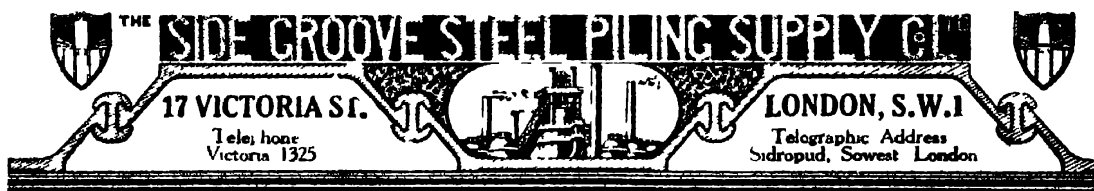
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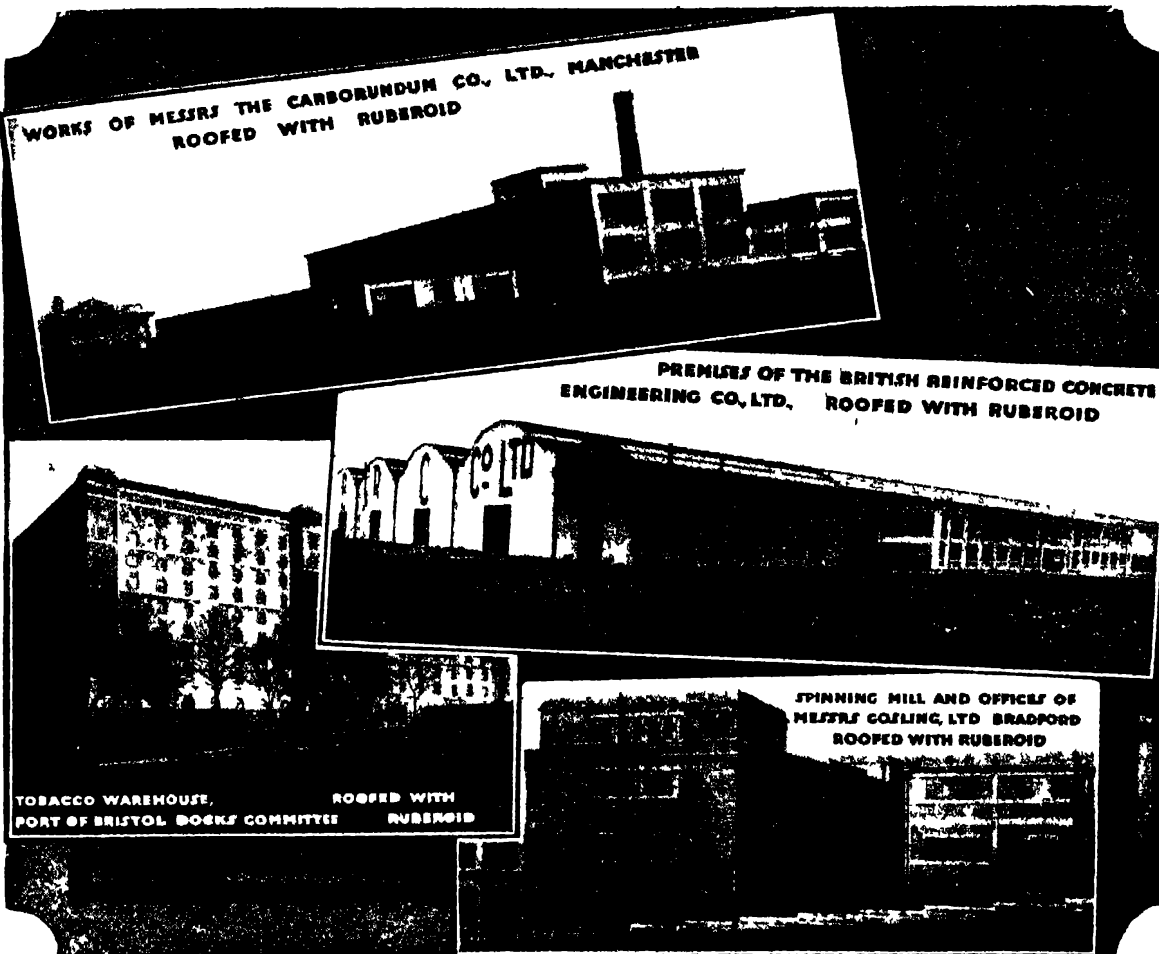
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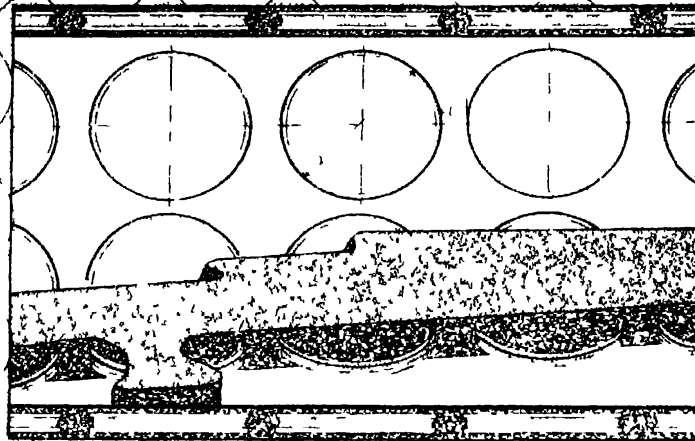
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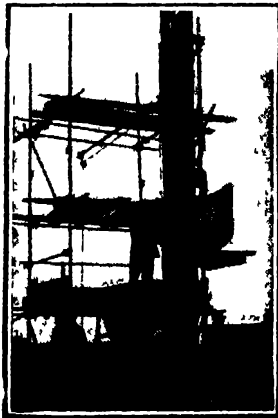
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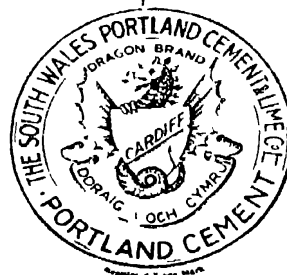
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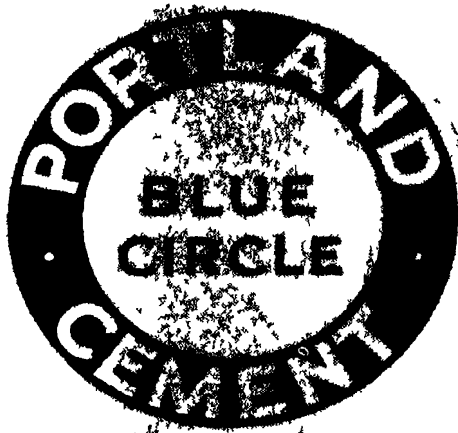
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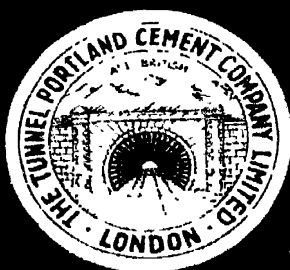
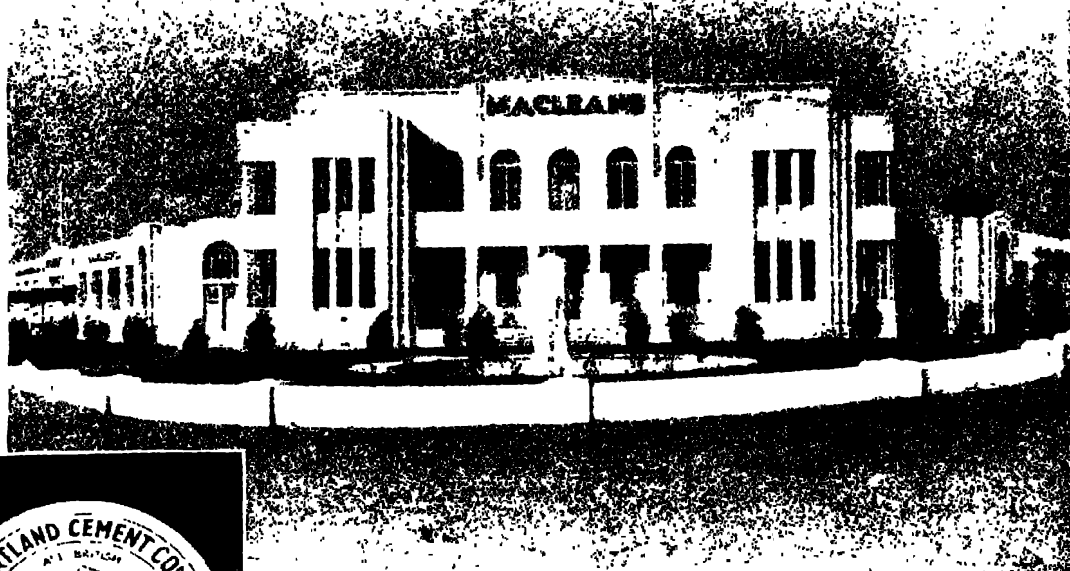
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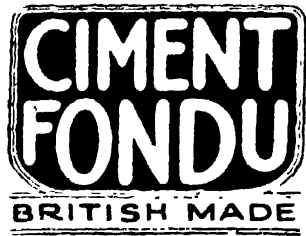
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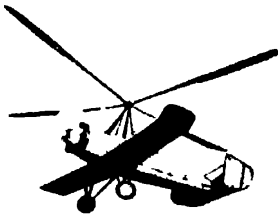
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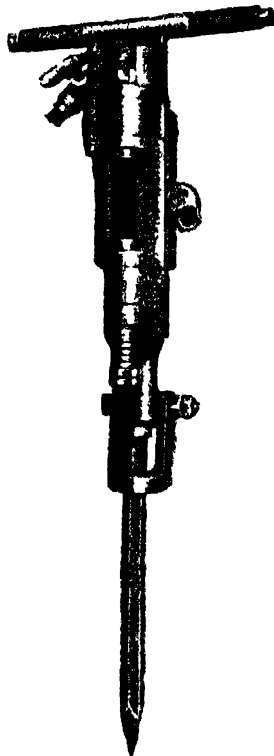
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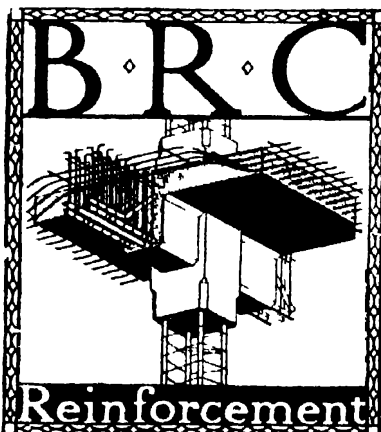
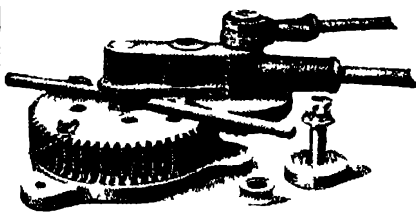
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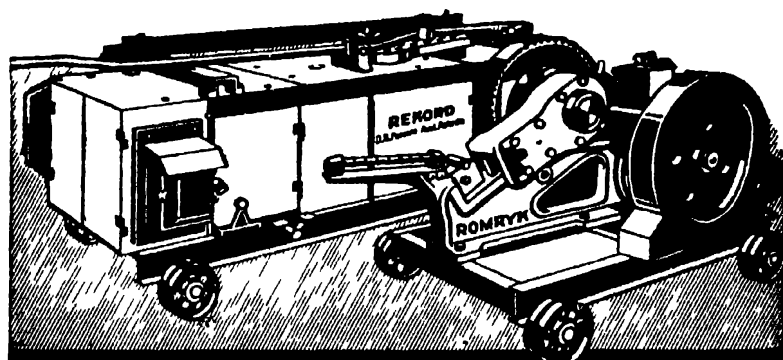
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Concrete Cottages

"CONCRETE COTTAGES, BUNGALOWS, AND GARAGES." By ALBERT TAKEMAN, I.R.I.B.A., M. Inst. C.E. Re-written 1932. 130 pp., 86 illustrations. Price 55/-, by post 55/- 6d.

Several designs for cottages, bungalows, and garages have been specially prepared by the author for construction in concrete, and with each design is given all the information necessary for its erection, including Perspective Drawings showing the same designs with flat and pitched roofs, Plans, Sections, Full Working Drawings, and Detailed Bill of Quantities.

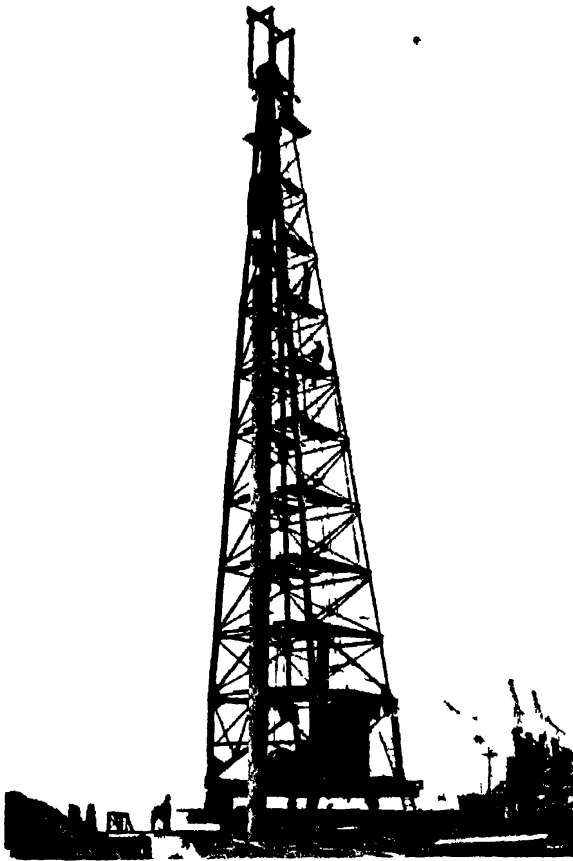


Illustration of 5-ton 90-ft Frame
with Monkey on Thames-side job

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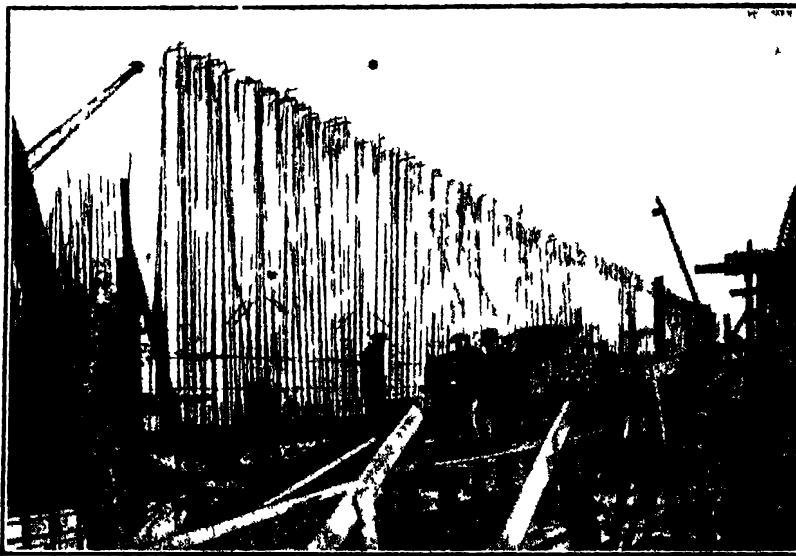
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CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume XXVIII No. 9

SEPTEMBER, 1933.

EDITORIAL NOTES.

The Training of a Civil Engineer.

WHILEAS engineering was at one time divided into two branches only—civil and military—there are now so many subdivisions that the training required for any one is specialised to a large degree. There is also a considerable amount of overlapping between certain branches of engineering work, and a definition which covers the duties of a civil engineer is difficult to obtain. The design and construction of railways, docks, harbours, canals, and hydro electric works are some of the subjects with which civil engineering deals, but even among these there are some portions in which collaboration with mechanical and electrical engineers is necessary.

In this country a recognised qualification of a civil engineer is membership of the Institution of Civil Engineers. In consequence a guide to entry to the profession should be based on the requirements of this body. Those who wish to obtain the diploma of the Institution cannot do better than consult a book * which has recently been published, and whose authors are two well known consulting engineers. The authors are strongly in favour of the pupilage system, but at the same time point out that the Institution does not make this compulsory for admission. We agree that the experience gained during a paid appointment may not be all that is desirable when considered as engineering training, but we differ from the implication in the authors' statement that. In circumstances which created the necessity for such a paid assistant, it is highly improbable that any boy would receive the personal attention from his chief which is so desirable a feature in the case of the training of a civil engineer. There may be cases in which the pupil receives the personal attention of his chief, but it is probable that these are rare except in small firms. Too often pupils are handed over to the care of assistants, and it may be that some do not see their chief from one year's end to the other. It is often forgotten that the arrangement by which a young engineer becomes a pupil is a contract under which both parties have obligations. In the majority of cases pupils are well looked after and assisted in obtaining practical knowledge, but on the other hand there are instances where engineers who cannot provide adequate opportunities for acquiring practical experience receive pupils into their offices. Cases, also, are not uncommon in which the engineer—being a paid servant of a public body—is expected to take pupils with the object of obtaining cheap labour on his staff or as a monetary recompense for the inadequacy of his own salary. At the present time there is no remedy for

* "The Training of a Civil Engineer" By C. I. Howard Humphreys and G. Howard Humphreys, M.A. Inst. C.E.
(London: Edward Arnold & Co. Price 5s. net.)

these abuses. The position is such that two men of equal mental ability may leave the university simultaneously and enter pupilage, paying equal premiums, and yet one may obtain valuable experience on important works and, more important still, make valuable friendships, while the second may spend a large portion of his time in plotting small surveys required for law cases, checking bills of quantities, and other such work. We do not mean to imply that the latter class of work is not of importance, but it is definitely waste of a pupil's time when it can be done by a junior draughtsman. The pupil is in the office to learn civil engineering, and in the short time at his disposal should be given an opportunity of coming into contact with as many as possible of the duties which he may ultimately have to carry out. Pupilage is not the time for exercises in arithmetic.

Outside Great Britain pupilage, whether subsequent to a university course or as a complete training by itself, is not a recognised method of entering the engineering profession. In fact we are unaware of any country where this form of training is in vogue, and there is at least one country where it is illegal. The practice in the United States, for instance, is for the young engineer leaving the university to obtain a position with an engineer with whom he obtains his practical training. In some of the Dominions the same method of entry to the profession also exists, and the young engineer is paid a salary from the beginning of his career.

The numerous discussions which have taken place on the education of engineers are evidence that there is a considerable difference of opinion regarding the correct course to be pursued, and although we have drawn attention to some of the disadvantages which are associated with the pupilage system, this has chiefly been done in the hope that some of its abuses may be removed. It is known that some engineers require the student to have taken a high place at the degree examination before they will receive him as a pupil, but there is no means by which the pupil or his parent can estimate the engineer's ability to supply the necessary practical training.

The young engineer entering upon his practical training will find much useful information in the book mentioned, of particular interest being the chapters dealing with drawing-office practice and the duties of a resident engineer in charge of a contract. Another chapter contains a discussion of the appointments which are normally open to civil engineers—at the present time there is unfortunately a surplus of applicants for these—and the salaries which are generally offered. The authors include a useful list of textbooks which “ can be bought with absolute safety ”; to this “ The Training of a Civil Engineer ” might be added.

A Seaside Pavilion in Reinforced Concrete.

In this issue we illustrate the designs awarded the first and second prizes in the “ Concrete and Constructional Engineering ” competition for reinforced concrete designs in which prizes are given each year by the proprietors of this journal for the best design for a building in reinforced concrete by the atelier students at the University of London School of Architecture. The competition is set by Professor A. E. Richardson (the Head of the School), and this year the subject chosen was a seaside pavilion. The first prize was awarded to Mr. E. C. O'Farrell and the second to Mr. A. P. Ciregna. The jury consisted of Mr. Arthur Davis, F.R.I.B.A., Mr. Lovett Gill, F.R.I.B.A., Mr. H. V. Lanchester, F.R.I.B.A., and Mr. Corfiato and Mr. Collins of the Engineering Department of the University*.

The Foundations of the Ford Motor Company's Power House at Dagenham.—III.

By R. V. ALLIN, M.Inst.C.E.

Manufacture of Piles.

THE piles were specified to be of one part of aluminous cement to two parts of washed sand up to $\frac{1}{4}$ -in. mesh with not more than 4 per cent. of silt and four parts of crushed and washed $\frac{3}{4}$ -in. to $\frac{1}{4}$ -in. ballast. They were cast on four 80-ft. by 32-ft. pile beds covering a total area of 160 ft. by 70 ft., allowing a space



Fig. 21.

down the centre through which the mixed concrete was taken in train loads, consisting of a 24-in.-gauge petrol loco and two bottom-tipping skips, which were picked up by a 5-ton loco crane commanding the moulds.

The forms were erected on fixed timber platforms sunk into the ground and with a drained foundation. This platform consisted of 12-in. by 6-in. cross sleepers at 5-ft. centres, with 9-in. by 3-in. longitudinals spiked down to these sleepers at 2-ft. 8-in. centres, forming the ground shuttering for the bottom flats of the piles. These longitudinals were strutted laterally with $4\frac{1}{2}$ -in. by 2-in. stretchers. The side sections of the moulds were made in 15-ft. lengths, each side being removable when striking the moulds by slipping out the top tie bolts and slacking the folding wedges. These sections were re-used thirty-eight times during the period of manufacture of fourteen months. The rein-

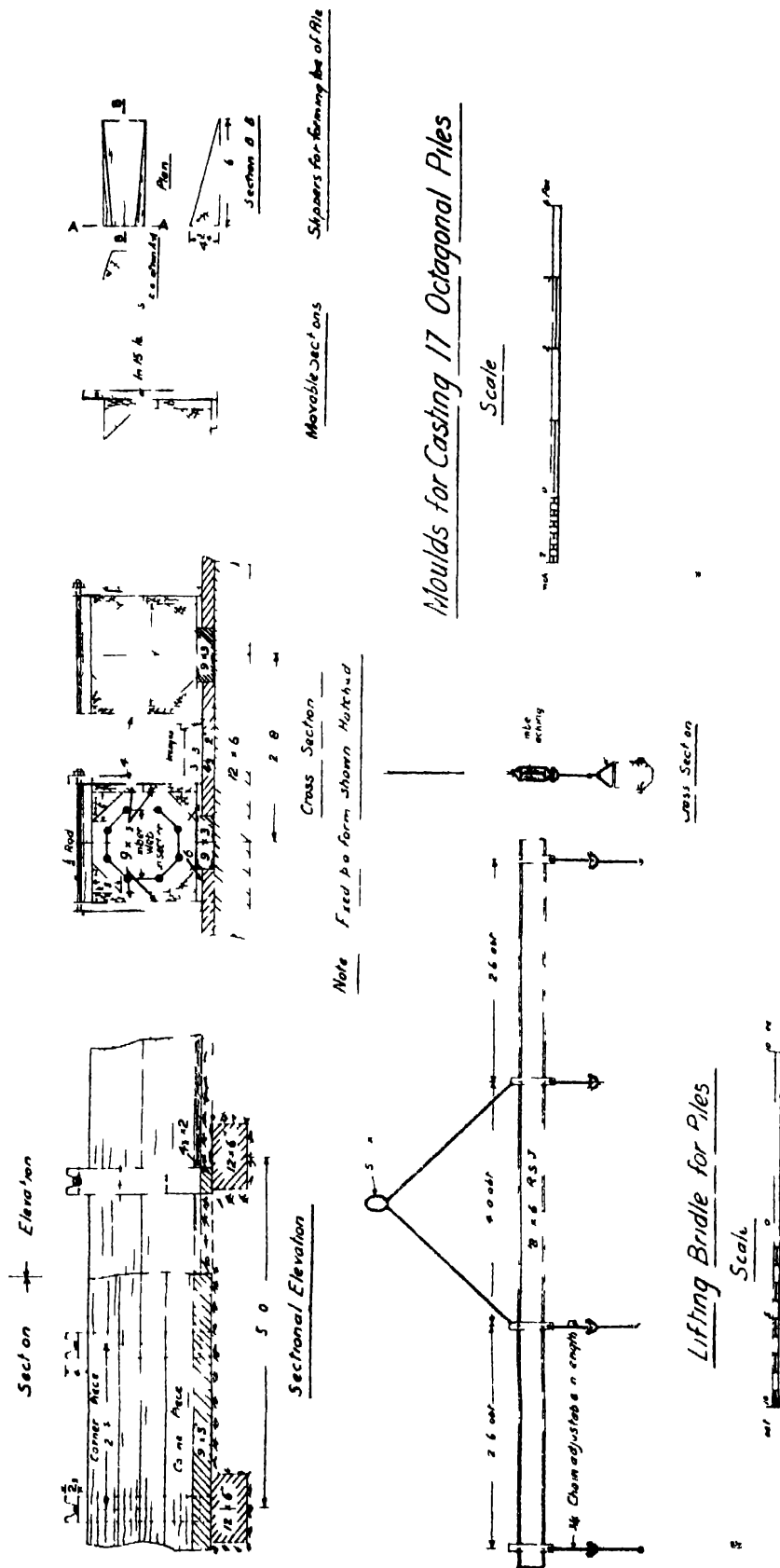


Fig 22

forcement was spaced with timber webs cut to take the bars and made in sections sufficiently small to allow of them being taken out of the cage before concreting (*Fig. 22*). This view also shows the lifting arrangements for four-point suspension of the piles. *Fig. 21* shows the setting up of the reinforcement prior to fixing the moulds and the longitudinal timbers forming the bottom flats of the piles, while *Fig. 23* illustrates the bottom-dumping skips brought up to the crane preparatory to pouring the concrete in the moulds. In *Fig. 24* is a general view of the north side of the pile yard with the crane striking the moulds of a freshly cast pile; this also shows the 15-ton electric derrick which picks up

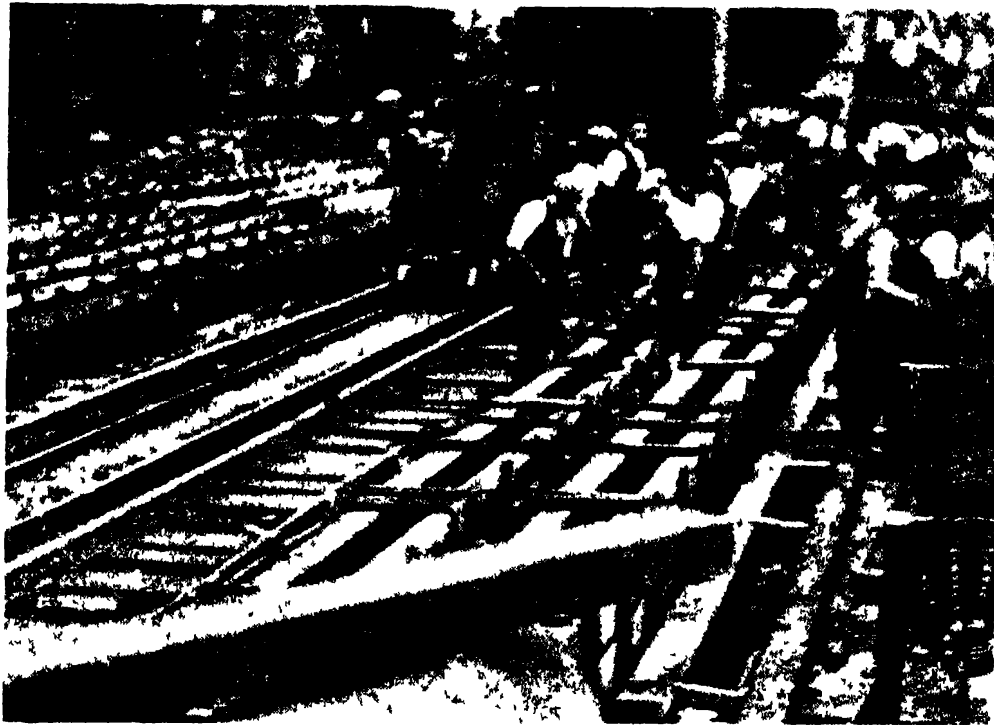


Fig. 23.

the piles from the casting beds and transports them to the stack and thence to the pile frame as required.

The sides of the moulds were struck from 4 to 6 hours after pouring, and it was specified that the piles might be moved 48 hours and driven 72 hours after pouring. It was found, however, that after a period of 60 hours the piles would usually withstand the hard driving. The piles were kept wet by sprinkling them for at least 24 hours after casting.

It was essential that the piles should be accurately cast at the head and shoe, as a head fitting tightly into the helmet led to spalling and a non-axial shoe to drift in the pile when driving. It was also necessary to ensure that no laitance should be left at the head, since this would lead to spalling in driving, and that there should be no "wind" in the length which might introduce bending stresses when driving.

The curing of the pile was of particular importance owing to the heat which developed in the setting of the aluminous cement concrete, and it was found to

have considerable influence on the strength of the piles. In connection with the curing it was found necessary to drain the pile beds thoroughly, as they were laid on soft clay.

The minimum crushing strength specified for the concrete of the piles was 4,000 lb. per square inch. Daily test cubes recorded strengths ranging from 4,090 lb. to 8,280 lb. per square inch after 7 days, with an average of 6,500 lb. per square inch.

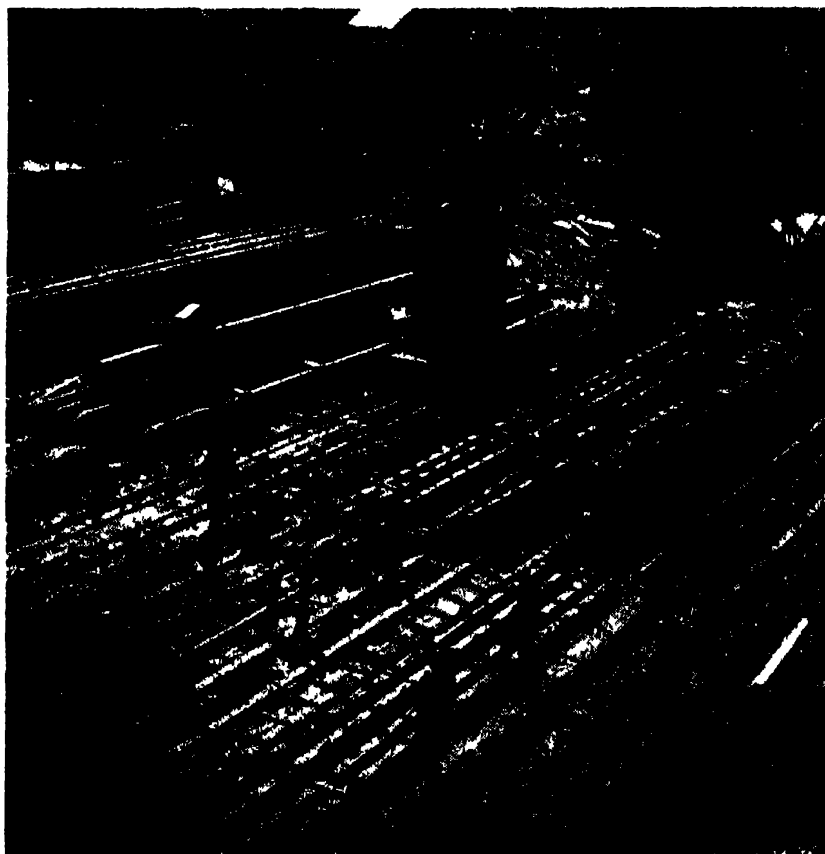


Fig. 24.

In the operations of making the piles, including unloading the aggregate at the mixer but excluding handling after casting, the following labour was employed: At the mixer station: 1 mixer driver, 6 labourers, 1 crane driver, and 1 banksman. In the pile yard—depositing and punning concrete: 1 ganger, 10 labourers, 1 crane driver, and 1 banksman. Setting steel skeletons: 6 steel fixers, with 1 burner and mate in attendance. Running concrete: 2 loco drivers and 2 rope runners. Repairing moulds: 2 carpenters. Watering piles: 2 boys.

The following is a list of the plant used in these operations: At the mixer station: One 3-ton steam crane and one $\frac{1}{2}$ -cb. yd. Fowler mixer (paraffin). At the pile yard: One 5-ton loco steam crane and one 15-ton electric derrick. Running concrete: Two petrol locomotives, 24-in. gauge.

The maximum number of piles cast in one 10 $\frac{1}{2}$ -hour net working shift was 19, representing 171 half-cubic-yard mixings.

The materials used in casting 2,045 17-in. octagonal piles were as follows:

$\frac{3}{4}$ -in. to $\frac{1}{2}$ -in. shingle, 7,660 cb. yd. ; sand ($\frac{1}{4}$ in. down), 5,360 cb. yd. ; aluminous cement, 2,240 tons ; chilled cast-iron shoes (80 lb. each), 73 tons ; $1\frac{1}{8}$ -in. steel reinforcement, 1,790 tons ; $\frac{5}{16}$ -in. helical binding, 277 tons ; $1\frac{1}{2}$ -in. pipe ferrules (17 in. long), 14,570 lin. feet ; 14- and 16-gauge binding wire, $4\frac{1}{2}$ tons. Consumable stores : Water, 860,000 gallons (approx.) ; steam coal, 320 tons, petrol, 2,500 gallons ; paraffin, 1,235 gallons, cotton waste, 290 lb. ; lubricating oil, 225 gallons, engine oil, 75 gallons ; black oil, 85 gallons ; oxygen, 60 cylinders ; acetylene, 8 cylinders ; Stauffer's grease, 200 lb.

The proportions of the aggregate used were 0.829 parts of shingle, 0.580 of sand, and 0.222 of aluminous cement. In making the piles an excess of sand was used forming a 1 $2\frac{1}{2}$: 4 mix, as it was found by experience to give more satisfactory results. Where less sand was used the piles showed a tendency to spall. The proportions previously given include a certain amount of waste of

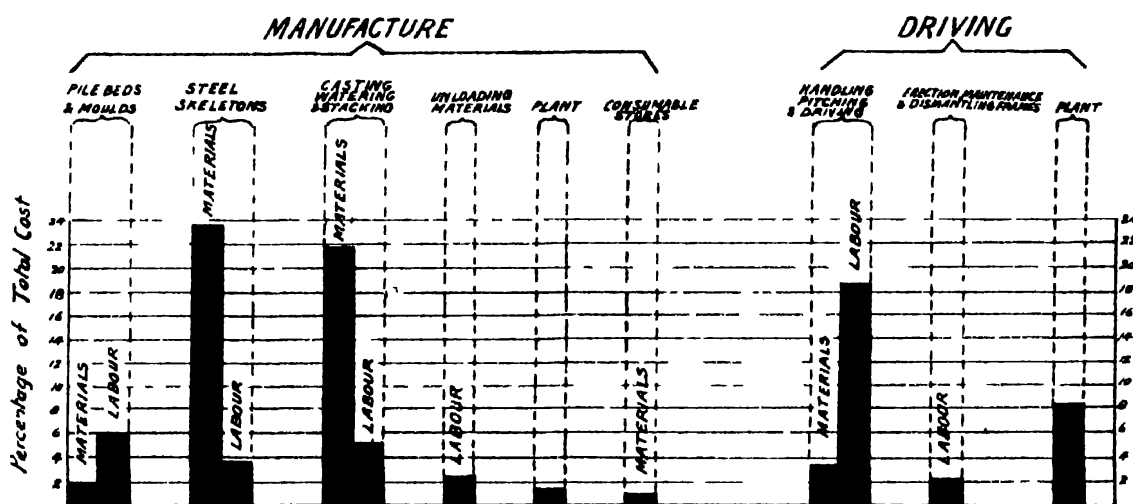


Fig. 25. Analysis of Expenditure on Piling.

the aggregate in transit. Fig. 25 shows an analysis of the expenditure on the manufacture and driving of the piles.

Turbine Basement.

The whole of the excavation for the turbine basement, pump house, and valve chamber was designed to be carried out within a cofferdam consisting of No. 3 Larssen steel sheet piles, driven down to - 40.00 O.D. except on the west side of the pump house, where No. 5 Larssen sheeting was used, and driven to - 55.00 O.D. as a safeguard against possible disturbance of the foundations there due to driving the circulating water tunnels which it was thought would reach this point after the foundations were completed. The cofferdam for the turbine basement was about 305 ft. long by 86 ft. wide at its widest point, and was divided into thirteen compartments (Fig. 26). Originally it was intended to carry the excavation in the open to a depth of about - 28.00 O.D. in some places, and it was evident that unless special provision was made large quantities of water would enter the excavation. Heavy pumping was inadvisable owing to the close proximity of existing buildings, and it was decided to treat the ballast below the foundations within the steel sheeting by cementation.

Cementation.

The depth and nature of the ballast and its suitability for cementation were ascertained by six core borings taken before work was commenced; in addition a number of confirmatory 2-in. flush borings were taken immediately in advance of the work as it progressed in order to explore possible local depressions of the ballast and to ensure that the cementation was not commenced at too high a level. These borings were carried out by hand and consisted of flushing (down to ballast level) 2-in. piping through which the water was passed to the shoe of the pipe. Samples of flushings were frequently taken during borings, and when a return of sand or gravel occurred it was concluded that ballast had been reached. It was proved by subsequent excavations that the observations so obtained were not very accurate and tended to show that the ballast appeared at lower levels than was actually found to be the case. It was specified that the ballast should be treated for a depth of 6 ft. below foundation levels, but this depth was later increased in places to 8 ft.

Cementation was carried out in the usual manner, hollow drill rods fitted at the bottom with perforated bits being driven with hand-operated percussive drilling machines mechanically rotated; when the bit reached the lowest required depth, the flushing water was replaced by cement-laden water which was forced under pressure through the perforations into the surrounding ballast. The cement mixture was pumped continuously during the lifting period throughout the specified depth. Alternatively, when trouble was being experienced with fast rods and leakage of cement, the usual cross bits were replaced by pointed bits which were "bumped" down instead of rotated, and cement was injected whilst drilling, commencing immediately the bit reached the top of the depth specified for treatment. These expedients were found to improve the progress of the work. The site for drilling was such as to permit of easy erection of two level working platforms. The top platform was 7 ft. above the lower, permitting the drilling to be commenced with a 10-ft. rod and carried down below the upper floor, whence it was continued and completed by the man below who used the same machine, which he then passed back to the upper level for the addition of the second rod.

The centering of the holes, which were 2 ft. 8 in. apart, was accurately done by boring 2-in. holes in 9-in. by 3-in. deals rigidly fixed over the lower platform. The "verticality" of the holes depended entirely on the care taken by the driller, as owing to the necessity of passing the drilling machine through the upper platform, satisfactory templates could not be fixed at the higher level. The average depth of ground penetrated to ballast level was 34 ft. and the average penetration into the ballast was 10 ft. 8 in. The average time taken per hole, including drilling, injection of the ballast, and the complete withdrawal of the boring rods, was 101 minutes. Each injection was carried to refusal within predetermined limits of pressure and quantity. The pressure varied, but in some instances as much as 500 lb. per square inch was developed by the pumps, but no external disturbance appears to have been incurred beyond the area treated.

The drilling and injection were carried out with seven François cementation units, and the average number in operation throughout the work was three. Each unit consisted of a cementation steam pump, a compressed-air drilling machine, and complementary equipment. The following plant was also em-

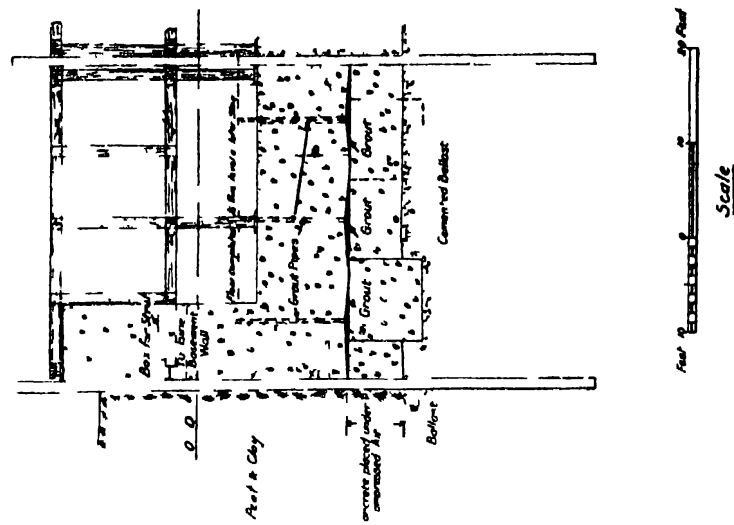
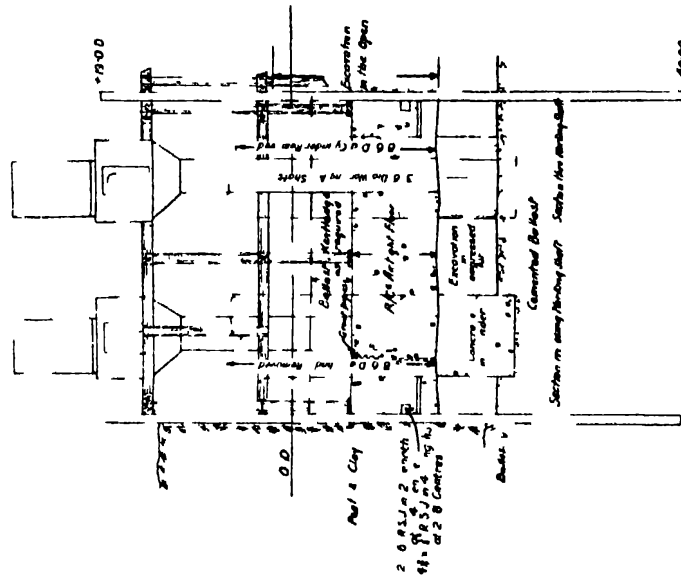
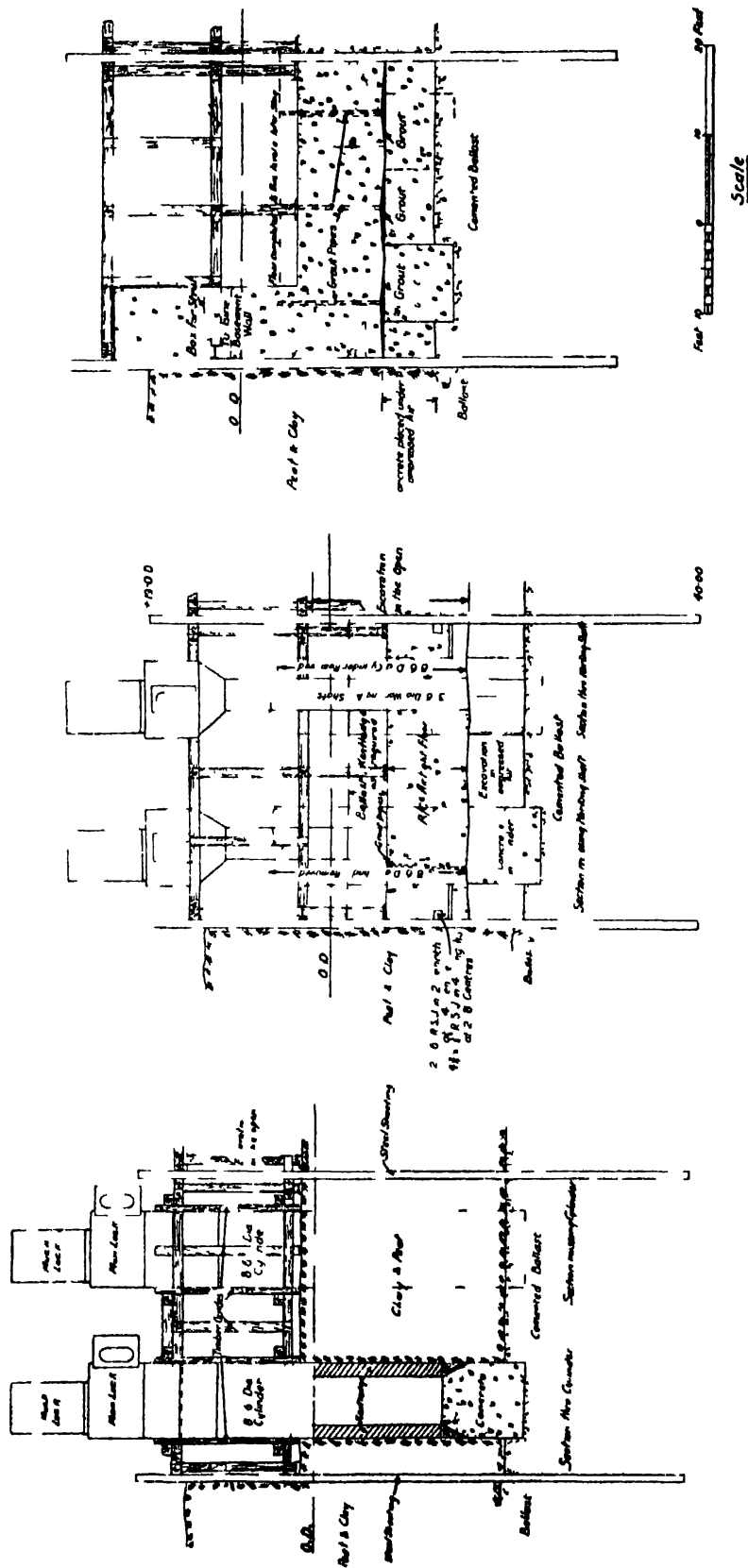


Fig. 27A.

Fig 27B

Fig. 27C.

EXCAVATION FOR SECOND FRAME AND
CYLINDERS SUNK AND SEALED

CONSTRUCTION OF AIR TIGHT FLOOR AND
EXCAVATION UNDERNEATH IN COMPRESSED
AIR

SPACE UNDER AIR TIGHT FLOOR CONCRETED
BALLAST KENTILLDGE REMOVED AND WALL
BUILT UP

FIG. 27C.

FIG. 47C.
SPACE UNDER AIR TIGHT FLOOR CONCRETED
BALLAST KENTLEDGE REMOVED AND WALL
BUILT UP

Construction of Typical Compartment in Turbine Basement

ployed Two oil-driven compressors of a total capacity of 380 cb ft. of air per minute at 80 lb per square inch pressure, one petrol-driven compressor with a capacity of 210 cb ft of air per minute, and two portable steam boilers. The approximate quantity of the work involved is indicated by the following figures: Area treated, 2,359 sq yd, volume of ballast treated, 5,240 cb yd, number of holes drilled, 3,300, total length of drilling, 147,000 lin ft, quantity of cement injected, 1,405 tons, water used for all purposes, 3,400,000 gallons.

Excavation was commenced in compartments Nos 3, 4, 5 and 7 (*Fig 26*). It was found that compartment No 3 could be bottomed up in the dry, but in Nos 4, 5, and 7 the cementation was not sufficiently impervious to prevent the inflow of a large quantity of water accompanied by fine sand. These compartments were allowed to fill with water, the excavation was completed, and the concrete seal placed with the aid of divers. When the "blows" occurred the tide level was about 34 ft above the bottom of the excavation. Owing to the vital importance of avoiding any withdrawal of the sand from the surrounding foundations previously mentioned it was considered necessary to review the design and methods of construction of this portion of the work. The modification of the design has been described in Part I of these articles.

Modified Construction

The position of the construction when these difficulties occurred was that in compartments Nos 6, 8, 9, 10, 11, 12, 13, 14 and 17 (*Fig 26*) practically no work had been carried out other than their enclosure by sheet piling, sufficient excavation to fix one frame of timber, and cementation of 6 or 8 ft of the underlying ballast stratum. It was decided to carry out the lower excavation in these compartments by compressed air which would completely abolish pumping. The following method for completing the foundations was adopted.

Excavation was carried down to allow the fixing of a second timber frame and steel cylinders of 8 ft 6 in internal diameter were sunk (partly in free air and partly in compressed air) in positions in the compartments which interfered as little as possible with the timber already in position, advantage of which was taken to guide the cylinders. These cylinders were sunk by internal kentledge to a good foundation of cemented or partially cemented ballast, and the lower parts of the cylinders were filled with concrete (*Fig 27A*). The excavation in the open was then proceeded with to levels which it was judged would give a cover of about 6 ft of clay over the ballast and the cylinders removed to that level. Reinforced concrete floors 8 ft to 11 ft deep were then constructed resting on the cylinders and covering the whole area of each compartment. The ground shuttering for these floors was laid to longitudinal ridges and furrows with grout pipes to each ridge. Under these floors the remainder of the excavation for the foundations was to be taken out under compressed air, and they were therefore provided with air shafts and locks to give access underneath. It was arranged that any surplus of upward pressure of the compressed air over the dead weight of the floors and cylinders should be balanced by placing ballast kentledge on the top of the floors (*Fig 29*). The floors were therefore only reinforced to withstand such stresses as would occur if the air pressure failed. Compressed air was then applied and the remainder of the excavation down to a good foundation, was carried out (*Fig 27B*).

In the deeper compartments the excavation was taken out in trenches about 6 ft. wide around the inside of the steel sheeting; these trenches were bottomed up and concreted in lengths and the "dumpling" was taken out afterwards. Where considered necessary the sheeting was struted back to the cylinders, but as it had the support of the pressure of the compressed air, which varied between $\frac{3}{4}$ and $1\frac{1}{4}$ tons per square foot, the struts were only employed as a safeguard in the event of failure of the air pressure. The space formed by the excavation was then filled with concrete which made, with the air-tight



Fig. 28.

floors and the cylinders, a solid mass from the floor of the basement down to the foundation selected (*Fig. 27c*).

In all the operations of sinking and cutting off the cylinders up to the time when they received the load of the air-tight floor, enough kentledge was used to balance a possible hydrostatic pressure corresponding to the highest recorded tide level of $+17.30$ O.D. (*Fig. 28*) and the cylinders were founded at levels ranging between -20.00 O.D. and -39.00 O.D.

Fifty-six cylinders were sunk in the nine compartments and the whole of the work was carried out without any incident calling for comment. In compartments Nos. 6 to 11 at the east end of the power house there were abundant evidences of the old breach referred to in Part 1 of these articles, and a pre-Elizabethan anchor was discovered at about -25.00 O.D. (31 ft. below shore level). For this reason it was decided to subject the cylinders in these compartments to a bearing pressure test by loading them with kentledge after they had been sealed and before the surrounding excavation had begun. In addition to the application of sufficient weight to overcome the estimated skin friction

of the surrounding material (2 cwt per square foot) the cylinders were loaded to give a pressure on the foundations of 5 cwt per square foot. The resulting settlements varied from nil to $1\frac{1}{4}$ in with an average of $\frac{5}{16}$ in. The settlements recorded for cylinders founded at higher levels tended to be less than for those



Fig 29.

founded at lower levels, due possibly in some measure to the containing effect of the sheet piling.

Air-tight Floor.

The lowest sections of the cylinders which were sealed with concrete were provided with splice bars to connect them to the air-tight floors. These floors were reinforced to withstand only their own dead weight and that of the ballast kentledge referred to. The amount of air pressure allowed for in providing ballast kentledge on the top of the air-tight floors was judged from the pressures

found necessary in sinking the cylinders, and varied between 12 and 19 lb. per square inch.

The floors were poured in lifts. The first lift was 4 ft. high and contained all the reinforcement. At a distance of about 2 ft. above the underside of these floors lengths of 3½-in. by 1½-in. rolled steel joists were welded on the Larssen steel piling at about 2 ft. 8 in. and at centres projecting 4 ft. into the concrete. The object of fixing these lugs was to check the tendency for a gap to be formed between the concrete and the sheeting (caused by either air pressure or shrinkage of concrete) which would have resulted in excessive loss of air. The lugs were found to be effective, as the leakage of air was negligible.

In compartments Nos. 6, 8, 9, 10, and 11, in order to distribute some of the weight of the foundations on the sheet piling, 12-in. lengths of 10-in. by 6-in. rolled steel joists about 1 ft. 4 in. apart were welded to the sheeting 3 ft. above the underside of the floors (*Fig. 7*).

The depth of the excavation under the floors varied from 7 ft. to 16 ft., and care had to be taken that the bottoms of the cylinders were not undercut. The excavation was carried out so that at no point should a line drawn from the excavation to the bottom of any cylinder have a steeper up-grade than one in ten. In the deeper compartments rolled steel joists were left under the shuttering of the air-tight floors before they were concreted, and subsequently used as walings against which the steel sheeting was strutted back to the cylinders. When filling with concrete the space under the floors and up to the roof, the bottom of the first lift was put in fairly wet to fill the inequalities in the ballast bottom. In packing up to the roof of the chamber the concrete was mixed stiff, but owing to the settlement a space was formed below the soffit. Grout was poured in the first instance down the working air shafts and had therefore a pressure due to any head to which it may have risen in the shaft in addition to the air pressure. Enough time was left for shrinkage of the cement and then a second pressure grouting was applied through the pipes in the ridges of the roof (*Fig. 27*).

(To be continued.)

International Road Congress.

THE next congress of the Permanent International Association of Road Congresses is to be held in Munich in September 1934. The address of the British Organising Centre is 7 Whitehall Gardens, London, S.W.1. Papers are to be read by Mr. H. E. Lunn, County Surveyor of East Sussex, Mr. J. E. Swindlehurst, Borough Engineer of Hampstead, and Mr. G. H. Whitaker, City Engineer of Cardiff, on progress in concrete roads; by Mr. H. T. Chapman, County Engineer of Kent, Mr. A. T. Gooseman, City Engineer of Leicester, and Mr. F. W. Smart, County Surveyor of Bedfordshire, on progress in the use of tar for road work; by Mr. D. Edwards, Borough Engineer of Brighton, Mr. A. C. Hughes, County Surveyor of Hampshire, and Mr. E. J. Stead, County Surveyor of Somerset, on the progress in the use of bitumen for road work; by Mr. G. S. Barry, County Surveyor of Ayrshire, Mr. S. G. Stanton, Borough Engineer of Southampton, and Mr. J. P. Wakeford, Borough Engineer of Stockton-on-Tees, on the use of emulsions; by Mr. O. Cattlin, Borough Engineer of Lambeth, Mr. E. H. Colcut, County Surveyor of Cornwall, and Mr. T. Somers, City Engineer of Glasgow, on road surfaces; by Mr. H. H. Humphries, City Engineer of Birmingham, and Mr. A. J. Lyddon, Ministry of Transport, on the relationship between road surfaces and traffic; and by Mr. G. E. Ashforth, County Surveyor of Cheshire, Mr. C. L. Howard Humphreys, and Mr. E. S. Perrin, Ministry of Transport, on regulations governing road vehicles. Papers are also to be read on the safety of traffic.

Railway Viaducts near Belfast.

MAIN line trains of the L.M. & S.R. (N.C.C.) from Belfast to Londonderry and Portrush have to reverse at Greenisland, about 6½ miles from Belfast, thus involving delay to the trains and additional expense, owing to the necessity of maintaining the necessary staff. With a view to providing an improved service to Portrush and securing substantial economies it was decided to construct a loop line whereby trains can run straight through from Belfast to the north. This work is being carried out by the railway company in collaboration with the Government of

crosses the glen on a line approximately parallel to the existing structure. The second viaduct, known as the down-shore viaduct, is about 400 ft long, and the up-shore line is carried on the existing masonry arch. The undercrossing at the Belfast end of the main line viaduct has a skew angle of 16 deg., and is also constructed in reinforced concrete.

Both the down-shore and main-line viaducts are similar except as regards the width, the number of arches, and the height of the piers. The former consists of one main open-spandrel parabolic arch



Fig. 1.—General View of Viaducts.

Northern Ireland as an unemployment relief scheme.

The new line is about 2½ miles long, and crosses several roads and numerous streams. All bridges over and under the line are being carried out in reinforced concrete. At a point about three-quarters of a mile beyond the commencement of the new loop line the existing railway crosses a deep glen by means of a two-span masonry arch. The new loop line is about 200 ft. farther upstream and, crossing at an angle, requires a viaduct about 630 ft. long. At the Belfast end of this structure the line to Larne passes underneath that to Londonderry and

with a clear span of 89 ft. and a 30-ft rise; this is flanked on each side by three small filled-spandrel arches of 35 ft clear span and 11 ft 6 in. rise. The main-line viaduct has three similar large arches with five small arches at the Londonderry end and two small ones at the Belfast end which connect with the undercrossing. General views of the viaducts are shown in *Figs. 1 and 2.*

The down-shore viaduct, which carries a single pair of rails, is straight; the rail level is horizontal and is about 40 ft. above the level of the stream. The arches are 17 ft. wide. The main piers are carried down to a foundation of sand-

stone, while the piers of the small arches are supported on separate footings on boulder clay. Considerable difficulty was experienced in excavating for these piers as they are in the side of the existing railway bank, and it was necessary to maintain traffic while work was in progress. The piers are 6 ft. thick at the springings, have a batter of 1 in 60, and are heavily reinforced to withstand the large moment induced by unequal live loading

for each arch parallel the piers are trapezoidal. The large piers are built of mass concrete and are supported on spread footings 35 ft. by 39 ft. resting on boulder clay overlying firm sand. The piers at each end of the large spans are constructed as abutment piers.

The arch ring of the 89-ft. arch is 2 ft. 6 in. thick at the crown and 5 ft. thick at the springing, and is reinforced with 1½-in. diameter bars at 6-in. centres at the springing and 1¼-in. bars at similar



Fig. 2.—Plant View.

The main-line viaduct is on a 60-chain curve and has a rising gradient of 1 in 75. The large arches have a maximum height of 70 ft. above stream level. This viaduct carries a double line, and the arches are 29 ft. wide. The small piers are similar to those on the down-shore viaduct, except that they are much higher and much more heavily reinforced. They are supported on spread footings 22 ft. by 35 ft. carried down to good boulder clay. As this viaduct is on a curve and it was desirable to have the opposite springings

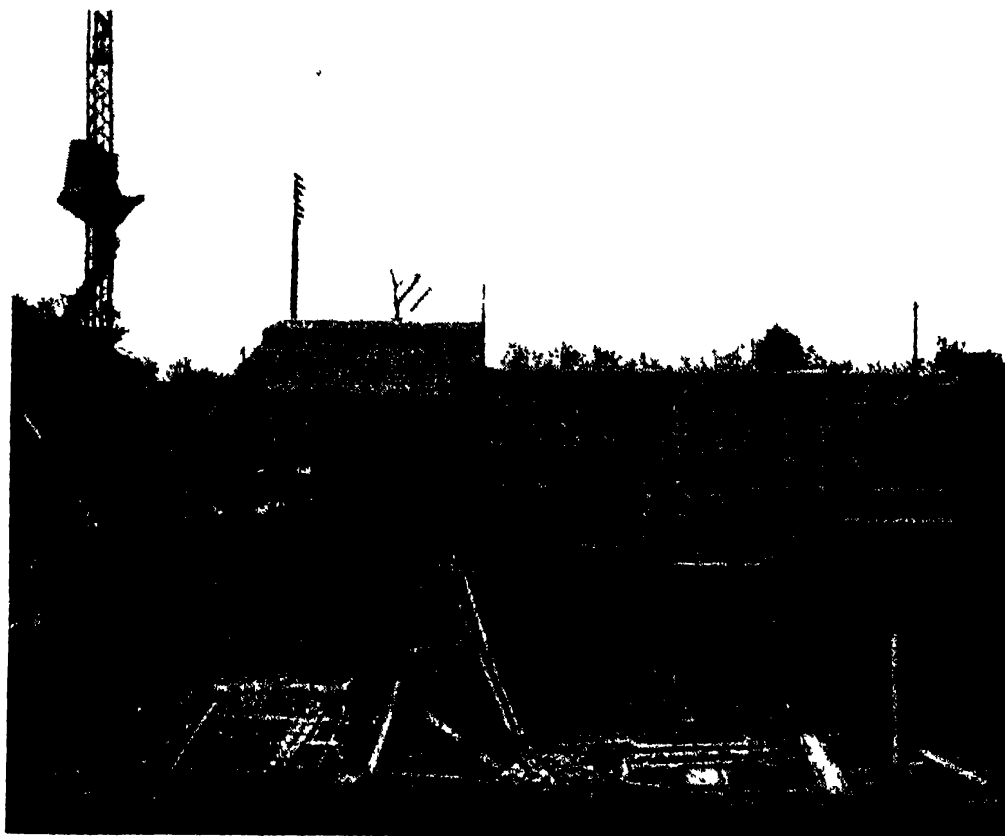
centres at the crown. The cross-walls are 2 ft. thick and reinforced with ¾-in. bars at 12-in. centres. These walls are spaced at 12-ft. centres and carry a floor slab 14 in. thick reinforced with ¾-in. bars at 8-in. centres. Sliding bearings are provided where the floor slab rests on the piers. The 35-ft. arches are 18 in. thick at the crown and 2 ft. 6 in. thick at the springing, and are reinforced with 1½-in. bars at 6-in. centres at the springing and ¾-in. bars at the crown.

The spandrel walls have a maximum

thickness of 2 ft. 6 in. at the springing and a uniform batter to 1 ft at rail level. Some mass concrete filling is provided over the tops of the piers, while the remainder of the filling is rough stone and ballast. A 2-ft. thickness of stone ballast is provided over the deck of the main arches.

Both structures have been designed to carry the British Standard Load for Railway Bridges (18 units) with the standard

plant. Advantage was taken of the existing viaduct, and a three-compartment bin was built spanning the stream at this point so that materials could be brought in by rail and unloaded directly into the bins. The bins are fitted with volume batchers leading into a hopper which in turn can be arranged to feed either of two 10-cb. ft mixers (*Fig 3*). The cement, which is rapid-hardening, is delivered in paper sacks and unloaded



• **Fig. 3.—Central Mixing Station, showing Bins, Cement Shed and Mixers.**

impact allowance. The working stress in the concrete is 750 lb. per square inch in compression and 75 lb. per square inch in shear. The reinforcement is round mild steel to British Standard Specification No. 15, and was designed for a working stress of 18,000 lb per square inch.

Concrete Placing.

The total quantity of concrete involved is about 17,000 cb. yd. and, as it is spread over a considerable area, close attention was given to the arrangement of the

into a cement shed at rail level. Two bags of cement are required to each batch, and the material is conveyed to the hopper through an inclined pipe carried down the back of the bins. The mixer operator controls the admission of the aggregates from the mixing platform. The cement batch is handled by men in the cement shed who release the batches in accordance with signals from the mixer operator. The concrete is mixed for a minimum time of one minute, which is controlled by a batch meter, and is discharged into side-tipping trucks which

are taken to the placing masts. Water is pumped from the stream to a storage tank, and from there is led to a service tank supplying the tanks on the mixers. These tanks are set for the requisite quantity for each batch, and the setting is only altered on the instruction of the engineer.

The down-shore viaduct was constructed first, and a $\frac{1}{2}$ -cb yd. mast placing plant covered the whole of the work

feed the other end mast by this method, and advantage was taken of an existing siding to deliver materials at this point. These were batched by hand and fed through another 10-cb. ft. mixer which discharged directly into the mast bucket. When the Londonderry end had been completed this mast was removed and re-erected at the Belfast end where it assisted in placing the concrete in the crossover. In this case also it was not

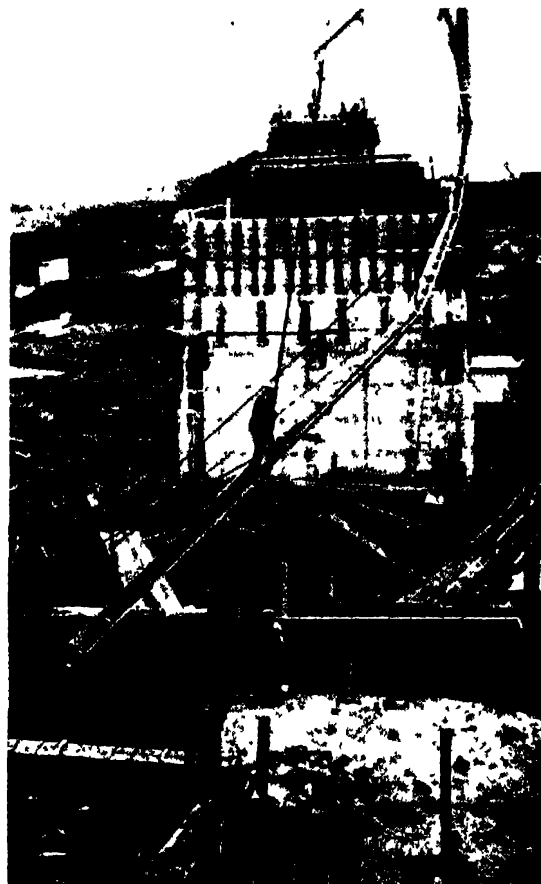


Fig. 5.- Looking North along Main Line showing Main Piers and Long Chute Line, suspended from Blondin Wire.

from two positions. In the main line viaduct three masts (*Fig. 4*, pp. 528-9) were used, a heavy mast 160 ft. high at the centre and two lighter masts about 100 ft. high at each end. A Blondin wire was suspended across the tops of these masts and used for slinging lines of chutes (*Figs. 2 and 5*). The concrete was conveyed from the central mixing plant to the centre and one of the end masts by means of power-operated inclined tracks. It was not possible to

possible to use the central mixing plant and the same arrangement was adopted as before. With the exception of this mast, the winch of which was steam driven, and the mixer, which was petrol driven, all the plant on the work was operated by electricity. Power was first obtained from plant installed on the site, but when the demand exceeded the capacity of this plant a supply was obtained from the Belfast Corporation.

Where placing conditions were fairly

easy it was possible to handle 10 cb yd hourly, but when much ramming was required the output was reduced to about 6 cb. yd hourly per mast. The total average weekly output was 500 cb yd. The longest line of chutes used was 230 ft., and it was found that concrete with

On the larger piers the boards were 8 in wide and on the smaller 6 in wide, horizontal chases were inserted at 4-ft. intervals. Advantage was taken of this to form the construction joints at these points, and the result is a structure singularly free from defects. The piers



Fig. 6.—Arch Centres, Main Line, 89-ft. span.

a slump of $2\frac{1}{2}$ in. could be placed effectively.

Surface Treatment.

As no money was available for special treatment of the concrete surfaces the most suitable way of avoiding a monotonous effect on these large areas became a matter of importance, and it was decided to leave the concrete as it came from the forms, with board marks visible.

were too irregular to make it economical to use standard steel forms, and as these would have had to be lined to give the effect desired timber was used throughout. Climbing forms were designed which enabled one set of formwork to be available for the full height of each pier. These forms consisted of $1\frac{1}{4}$ -in. tongued-and-grooved sheeting made into panels 4 ft. high and of convenient length. Double 7-in. by 2-in. studs were bolted

to the face of the previous lift and projected a distance of 4 ft. The panels were fastened to these studs, and $\frac{5}{8}$ -in rods threaded through strawboard tubes and screwed into 3 in by $\frac{5}{8}$ -in square plates were fixed to the top end of the studs. These were concreted in and used for securing the studs when moved up to the next position. The rods were then screwed out and the holes plugged with cement mortar. By this method it was

hinge at the crown and $1\frac{1}{4}$ -in tie rods. The shape of the arch was obtained by fixing timber nailing strips to the back of the trusses. The lagging was 6-in by 3 in in section. The spandrel wall forms, which had horizontal sheeting, were completed before the arch was cast and the portion in contact with the face of the arch was lined with $\frac{1}{8}$ -in Masonite Tempered Presdwood so as to provide a smooth face free from form marks.

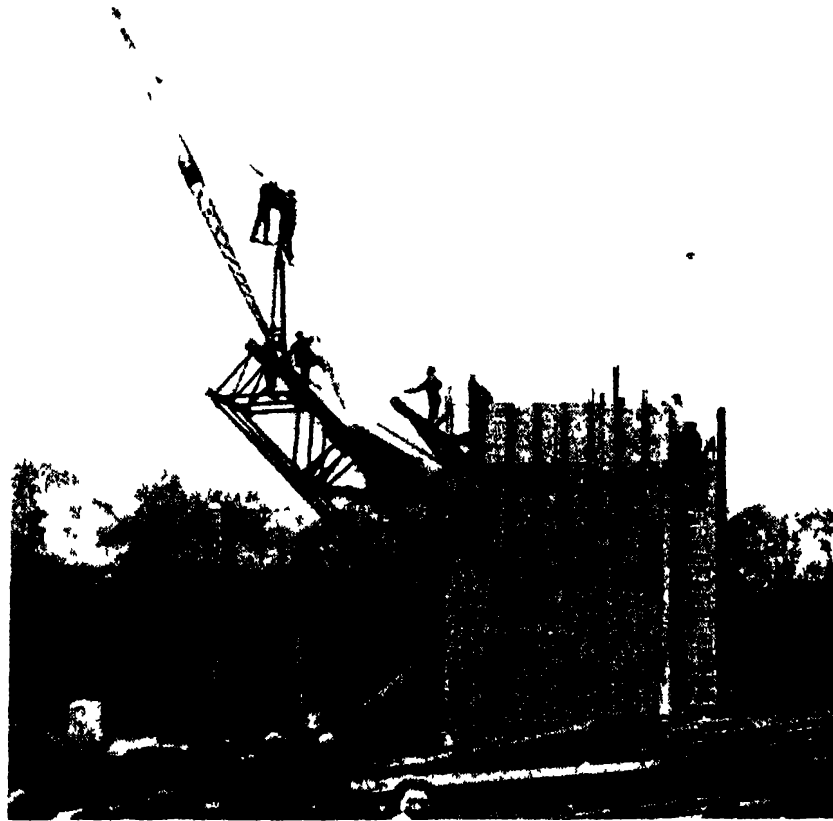


Fig. 7. Erection of 89-ft. Arch Centres on Main Line. Crane in position on nose of First Section.

possible to strip and re-erect a complete lift of formwork in one day. As many as fifteen uses were obtained from one set of forms.

Centering.

* As the arches were a considerable distance above ground level it was found economical and much quicker to use steel centres supported from the piers rather than to build up the usual type of timber trestle from the ground. In the small arches these centres consisted of steel trusses at 5-ft centres provided with a

The large arch centres (Figs 6 to 9) were also three-hinged trusses with $2\frac{1}{4}$ -in diameter tie rods. The trusses were spaced at 5-ft centres and both top and bottom chords were braced, while the 6-in by 4-in lagging was secured to nailing strips as in the case of the small arches. In the case of the down-shore viaduct the piers were erected to their full height before the arch was commenced and the centres were assembled on the ground and rotated into place about the lower hinge by means of tackle fastened to the top of the pier. Each

centre had a reaction of about 60 tons and was carried on cast-iron wedges. Extensions were provided to the bearing shoes, and the arch was struck by placing jacks under these extensions and transferring the load from the wedges to the jacks. The wedges were released by hand and the centres lowered on the jacks. The trusses were then moved sideways and dismantled by tackle slung from a cantilever beam fixed at the crown of the arch. In the case of the main-line viaduct the arches were cast in halves, three centres being used for one half, when this was completed the centres were moved sideways into position for the second half. In this case the piers were not completed before the erection of the

viaduct, and were jacked across a distance of 14 ft. 6 in. into their new position. They were then levelled, and the second half of the arch ring cast. On completion of this half the centres were lowered to the ground by tackle suspended from the underside of the arch ring.

In the down-shore viaduct the small arches were poured in one day and the centres struck in four days. On the main line the small arches were poured in two successive days and the centres struck on the fourth day, after the last section had been cast. In the large arches of both viaducts the arches were cast in five sections, poured on successive days, and the centres struck on the fourth day after the last section had been cast. The



Fig. 8. -Closing Centres of 89-ft. Arch on Main Line.

centres, and these were placed by a cantilever method. A small jib crane was mounted on top of the pier and placed the columns and beams and the bottom sections of the centres. These were tied back to the pier, and the crane was then mounted on the nose of the middle centre and the remaining sections of the centres lifted into place. This process was repeated for the other half of the centres, and the crane removed. The halves were thus in position, tied back to the piers, but were within 18 in. of meeting at the crown. The tie-backs were then eased until the crown hinges entered, tie rods were adjusted, the centres levelled on their wedges, and the lagging placed. The centres were struck by the same method as that used in the down-shore

maximum deflection at the centre of the large arches the day the centres were struck was $\frac{1}{4}$ -in.

Forms for the cross-walls and floor slab were made in panels similar to the formwork for the piers, and presented no difficulties.

The intersection has a square span of 15 ft. and an average skew span of 65 ft. The roof is a slab 15 in. deep reinforced with $\frac{1}{2}$ -in. rods at 9-in. centres spanning between cross-beams at 12 ft. centres. These cross-beams have a clear span of 15 ft. and are placed at right angles to the abutment. They have an effective depth of 3 ft 3 in., a stem width of 2 ft., and are reinforced with eleven 1-in. bars. Owing to the extreme skew of this crossing it was necessary to provide a parapet

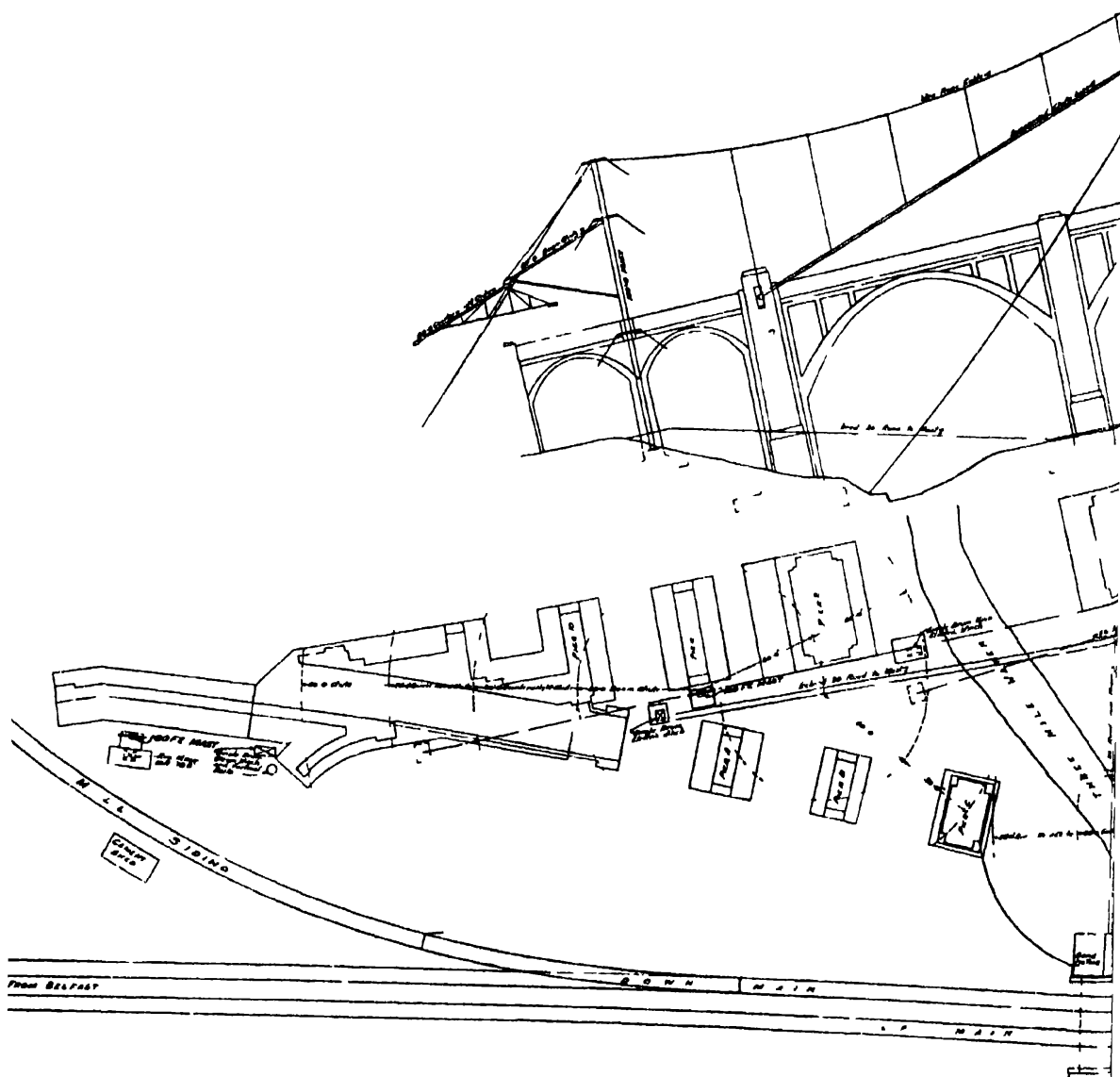
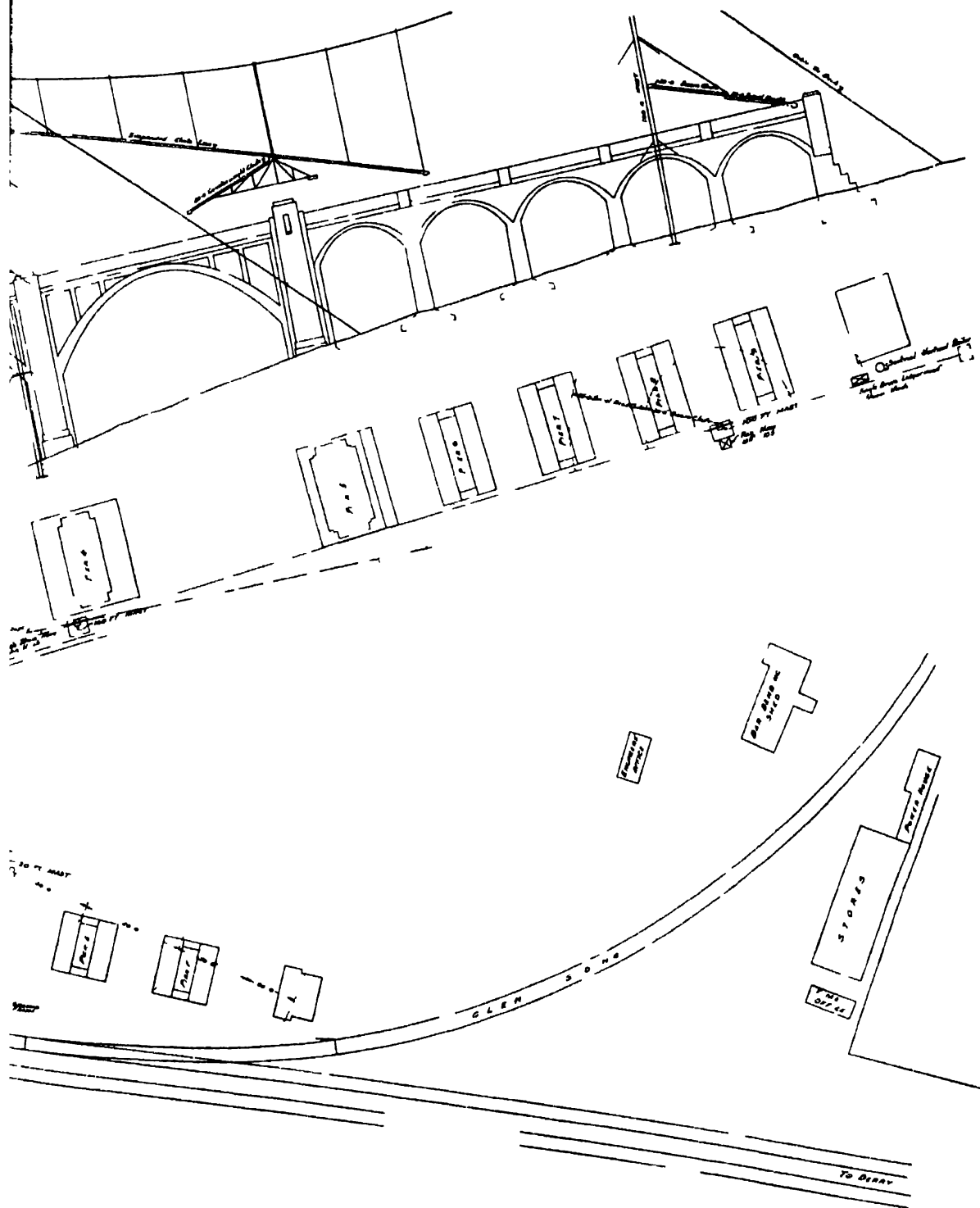


Fig. 4. Gen



at of Plant.

beam to carry the outer ends of some of the cross beams. This beam is of reinforced concrete, 10 ft. 10 in. deep by 20 in. wide and is reinforced with eighteen 1½ in. bars. One end of these parapet girders was fixed, and the other end was carried on a rocker bearing. The abutment walls were constructed in 4 ft. lifts with climbing forms similar to those used in the piers. The formwork for the floor slab was carried by rolled steel joists set in pockets on the abutment wall.

On the Ballast side of the crossover

parts of basalt screenings graded from ¾ in. to ¾ in. As it was impossible to secure washed sand at a reasonable price a washing machine with an output of 40 tons a day was installed. Considerable attention was devoted to the production of the concrete and the maximum slump permissible was 2½ in. It was found that this enabled the material to flow easily in the chutes and a dense concrete with a good face was secured. Six concrete cubes were secured for each day's run of each mixer and these were stored in a



Fig. 9 Main-Line, 89-ft Arch Centres moved to concrete second half of Arch.

there is a counterforted retaining wall 165 ft. long with a maximum height of about 29 ft. above foundation level. As there are new embankments on each side of this wall the lower part is slotted. The counterforts are 18 in. wide and spaced at 12 ft. centres, and the curtain wall ranges in thickness from 15 in. to 9 in. An expansion joint is provided at the centre of the wall.

Concrete Proportions.

All concrete was made with one part of cement to 2½ parts of sand and 3½

special shed with thermostatically-controlled heating. Notwithstanding these precautions it was found that owing to variations in the aggregate it was not possible to secure a concrete of uniform strength. The specification required a breaking strength of 2,000 lb. per square inch at seven days and 3,000 lb. at 28 days. These strengths were secured, but the range of the values was considerable. All the reinforcing steel, of which 700 tons were used, was bent on the site by a power machine.

The whole of the work is being carried out by the railway company by direct

labour recruited from the local labour exchange. Except on an earlier job of much smaller magnitude, neither foreman nor labourers had had any experience of this class of construction. Work is normally carried out in one 8-hour shift, but when it is not possible to pour the

quantity of concrete required by one lift overtime is worked as it was not considered desirable to introduce a second shift. Excavation for the first piers was commenced in February 1932, and the whole of the work was scheduled for completion last month.

Water Towers, Bunkers, Silos and Gantries.

THIS book,* the latest of the useful series on concrete and reinforced concrete published by Concrete Publications, Ltd., is a companion volume to "Reinforced Concrete Reservoirs and Tanks" by the same author, published in 1931. It is, however, self-contained and can be used without any reference to its predecessor so some repetition has been unavoidable.

The first part of the book deals in a thoroughly practical manner with the design and construction of the tank proper, the supporting structure being considered later. Several shapes are carefully treated, including rectangular, conical, and cylindrical tanks with flat, hemispherical, conical, and domed bottoms, and other more complicated shapes. Details of various methods of designing the bottoms, walls, and roof are given, with a consideration of their relative economy and with advice as to precautions to be taken to ensure watertightness. The effect of fixation of side walls by the bottom slab is treated in the way developed by Dr. Reissner.

A useful chapter is that dealing with the design of tanks for containing hot liquids. The author considers this problem in a sound manner, and points out that although the temperature gradient through a concrete wall may be found the stresses cannot be satisfactorily determined. The methods of stress determination proposed appear reasonable but, as the author points out, it is difficult to make a watertight concrete structure for large temperature ranges, especially where frequent alternations take place.

The second portion of the book concerns the design of bunkers, both shallow and deep. Such structures are very frequently met with in practice and the

requirements and complications of size, shape, and variety of load call for the exercise of much ingenuity, and there are many ways of treating the design of such structures. In many textbooks on design methods are shown of arriving at the pressures and loads of the contained material on the side walls and bottom of the bunker, but the student or designer is left to find out for himself the best practical way of designing the structure to take these loads. The author goes farther, shows the principles to use in designing bunker structures of various shapes and follows them up by practical illustrations.

Following naturally after bunkers is a section in which the design of silos for corn, cement, etc., is dealt with. This section is perhaps too briefly considered, and could with advantage have been illustrated by more examples.

Then follows a chapter on the design and construction of the legs or supports which are required for water towers, bunkers, and silos. The treatment of such structures in a strictly mathematical way is a matter beyond the practising engineer, so approximate methods based on reasonable assumptions justified by successful use in the past are here put forward. A useful addition to this section would be the consideration of the stresses in the sub-structure of a tank supported on columns with a central shaft and stairway.

Before the final section on the design of gantries there are several chapters dealing with matters of particular interest and use for the young engineer or non-specialist designer. In one of these chapters the author explains the abbreviated methods adopted by some reinforced concrete specialists in preliminary design and estimation of quantities. In the next particulars are given of special types of doors and feeding apparatus for

* "Reinforced Concrete Water Towers, Bunkers, Silos and Gantries." By W. S. Gray. 256 pp.; 170 illustrations. London: Concrete Publications, Ltd. Price 10s. (10s. 6d. by post).

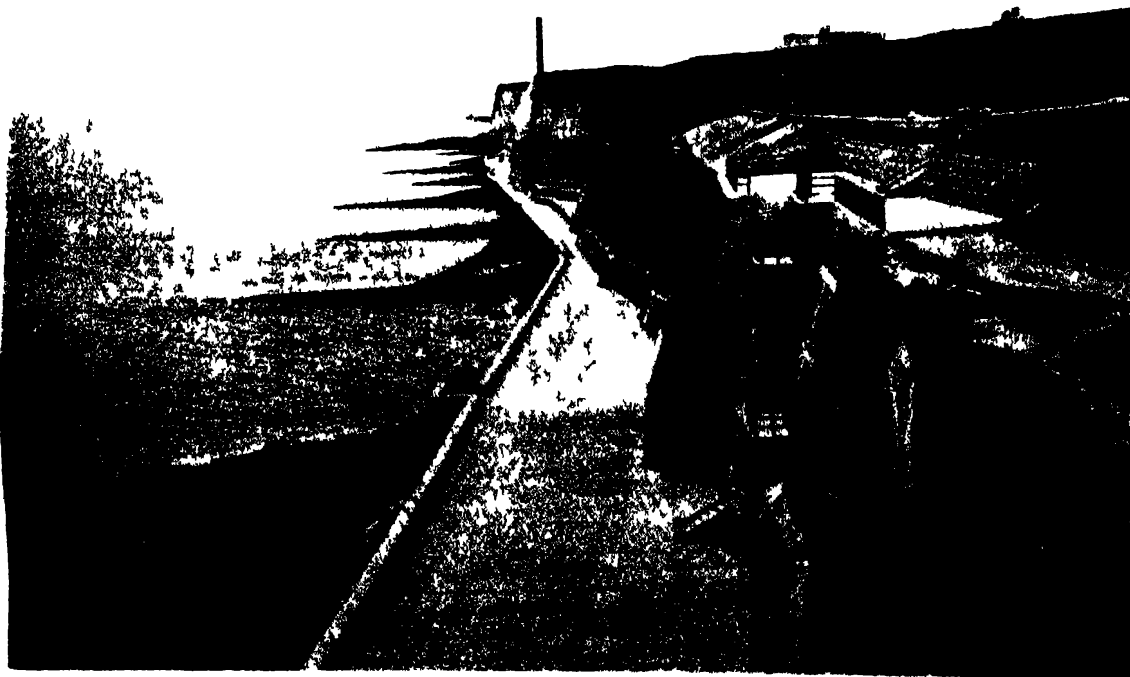
bunkers and silos. The author then devotes a chapter to the methods adopted by a contractor or specialist firm in the preparation of an estimate, the starting of the job and details of scaffolding and the placing of steel. These will be useful to the junior engineer though their application is not restricted to the structures specified in the title. This chapter also contains a recommendation that bending lists (of which a typical page is shown with the drawing to which it refers) should be supplied to the contractor. Such bending lists are not always supplied by designers and the work of taking off the steel is thrown upon the foreman or upon a member of the contractor's staff who cannot be expected to do this work so well as the designer or his draughtsman. Most of the specialist designing and contracting firms have found such steel sheets or bending lists ensure quicker and more accurate placing of the steel and prevent misinterpretation of the drawings.

The section on the design of gantries

is really too short to deal fully with a subject that one would rather expect in a book on bridges. This chapter contains some useful information which the author presumably felt might well be included as it does not appear elsewhere in this well known series of books.

The methods of design used in the book assume a knowledge of theory of structures and simple reinforced concrete but abstruse mathematics has been avoided. Common sense and a true perception of practical limitations of the material have been used. Among the very few statements which will not meet with approval is the recommendation to use expansion joints every 300 ft in gantries. So long a length of gantry exposed to the variations of temperature in this country would certainly crack. A more reasonable figure would be 100 to 150 ft at most.

The book is one which should be available in every designer's office for reference by the seniors and for study by the juniors. A. I. B.



New Promenade, Rottingdean-Saltdean.

The promenade and sea defence works illustrated were carried out this year under the direction of Mr D. Edwards M Inst CE, Borough Surveyor of Brighton. The promenade is surfaced with 6 in. of concrete reinforced with a single layer of B R C fabric.

A New Form of Reinforcement for Concrete.

(Contributed)

It is well known that if ordinary grades of steel are worked cold their tensile strength is improved: in other words, the steel is "physically developed." Some sixteen years ago a paper was read in London before the Society of Engineers describing experiments and tests on mild steel bars physically developed by twisting them about their longitudinal axis, and it was clearly demonstrated that the tensile

throughout its cross section. In the early experiments square and triangular bars were twisted. The results showed that the triangular section was not so good as the square section which in turn was inferior to a round section, the reason being that the fibres in the outer parts of the cross section of the bar, being more remote from the centre and thus having to describe a larger circle than the fibres

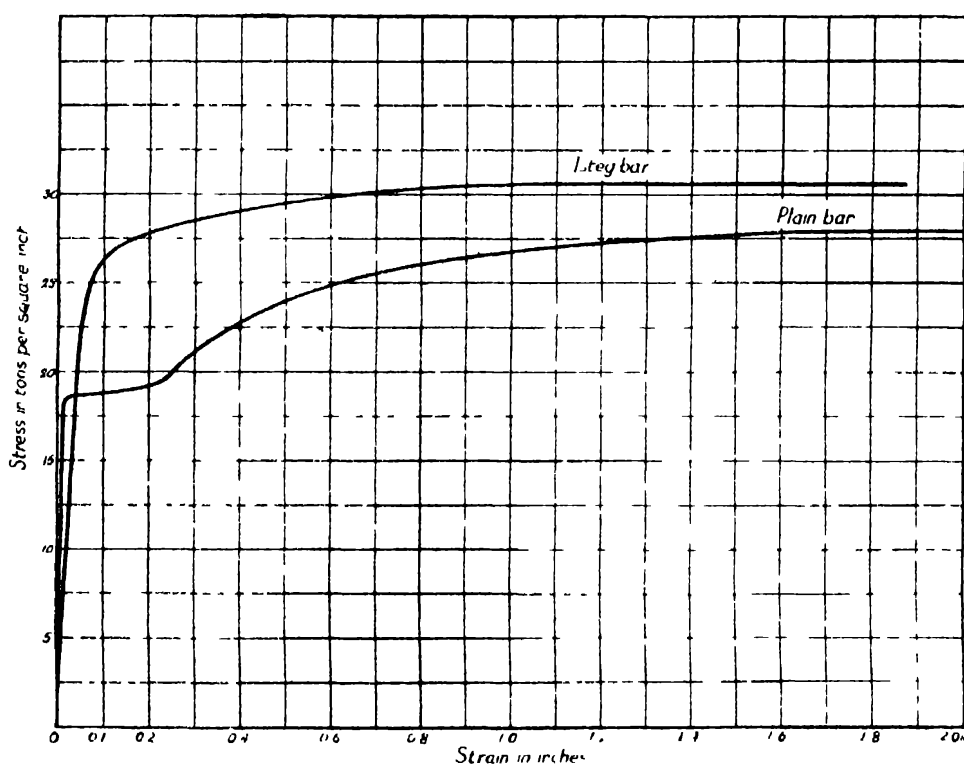


Fig. 1.

strength of the steel was materially increased.

The theory on which these experiments were based is sound, but when a single bar is twisted on its longitudinal axis no stresses are set up in the steel at the centre of the cross section of the bar (i.e. the axis of rotation), and hence there is no physical development at this point; also there is very little development in the area adjacent to the centre of the section, whilst at the outer edge of the section there is excessive development. Thus the bar is not uniformly developed

nearer the centre, became very brittle. In the case of the twisted round bar there is no visual indication of the amount of twisting the bar has undergone, hence the bars could easily be overdeveloped and rendered brittle; and also it would be difficult to distinguish such a bar from an untwisted bar.

Yield Point.

If we draw a diagram of stresses and elongations for several mild steel bars of the same manufacture we find that the yield points vary within wide limits and

that, generally speaking, the diagram is more or less erratic. This is explained by the relative lack of uniformity in commercial quality steel bars.

The yield point being much lower than the ultimate strength of the steel, in a reinforced concrete structure the tensile breaking strength of the steel does not

yield of a mild steel bar is considerably above this, consequently the ultimate breaking strength of the steel is never developed, since—long before it is approached—the relatively elastic steel has stretched more than concrete can stand.

Research and experiment in Central Europe have led to the development of

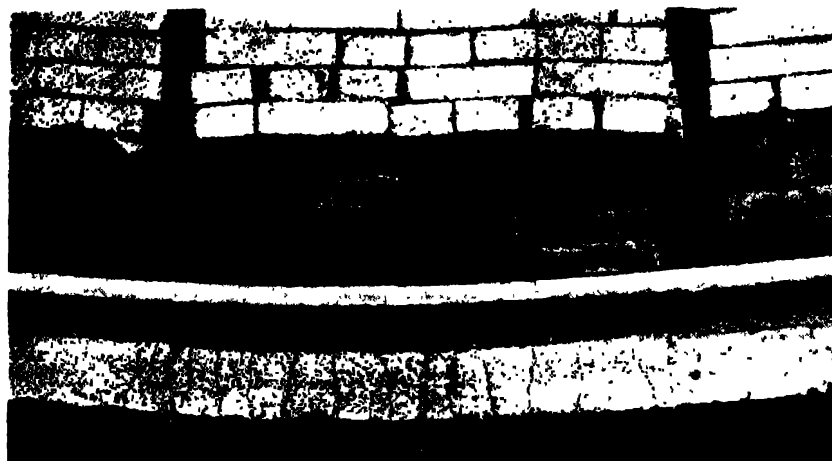


Fig. 2 (a).—Failure of "Isteg" Beam.

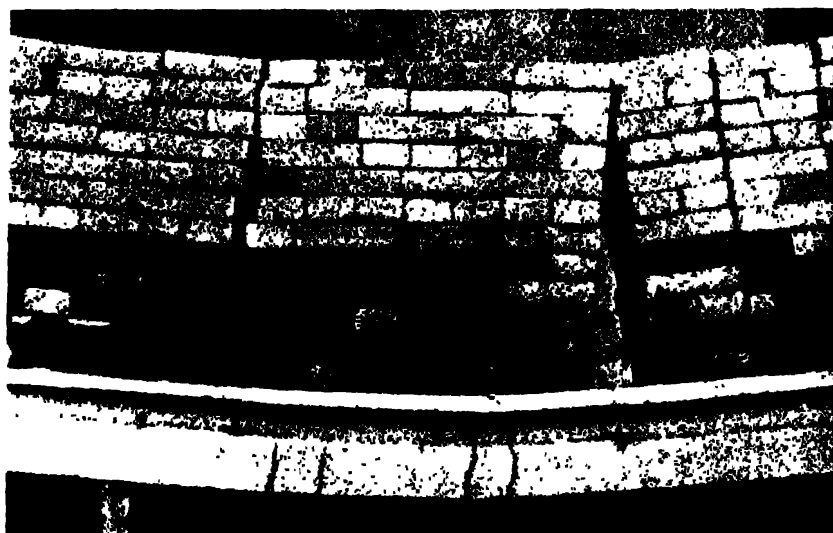


Fig. 2 (b).—Failure of Beam with Ordinary Steel Reinforcement.

come into question. Reinforced concrete is based on the co-operation of the two materials dependent on their proportional deformation. The steel can take up only a certain force resulting in an elongation with which the concrete can comply. It has been found that this elongation varies between 0.4 and 0.5 per cent. of the length of the steel rod. The normal

the "Isteg" process, which by a simple cold mechanical process alters the elastic behaviour of the rolled bar and corrects deficiencies of manufacture and rolling. Its point of departure is the stretching of the steel before its use. It has been found that if two round bars of ordinary commercial mild steel are placed side by side and stretched, while being twisted

together, to a predetermined point beyond the yield point of the component rods, the resulting twin bar, whatever the yield points of the component bars may have been, will have a regular stress-strain diagram (see *Fig. 1*) giving a yield point (taken at 0.2 per cent. permanent elongation) of a minimum of 58,500 lb. per square inch (see *Table II*). With this twin bar (known as "Isteg" steel) the axis of rotation is the line at which the composing bars make contact throughout their length—hence the entire cross section of each bar is developed. The amount of elongation or stretching

the section, have shown steel-breaking stresses exceeding 69,000 lb. per square inch.

With "Isteg" reinforcement the important factor, however, is increased safety. The steel is tested by the stretching and twisting process, each bar being submitted to at least twice the stress it would ever be called upon to stand in the concrete structure. The twisting has a levelling influence on the strength of the steel and softer parts of the bar are hardened, hence the regular stress-strain diagram. Bars with inherent defects, such as piling, seams, laps, etc (*Fig. 3*), which

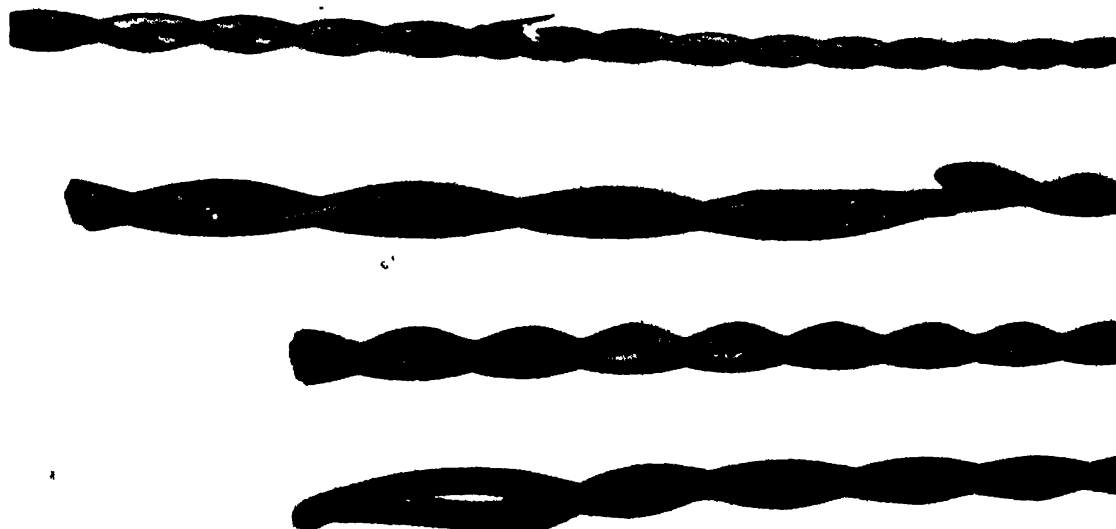


Fig. 3.

depends on the pitch of the spiral, which is regulated and can be checked at any time.

Assuming that an average concrete structure will withstand 0.4 per cent. steel elongation, over 80 per cent. of the breaking strength of the steel can be used. It is claimed that a structure reinforced with "Isteg" steel would not collapse owing to a sudden yielding of the steel, but due to a gradually increasing discrepancy between the deformation of concrete and reinforcement *Fig. 2 (a) and (b)*. Numerous tests of reinforced concrete beams, so designed as to collapse due to excessive steel elongation, that is with a reinforcement of less than 1 per cent. of

have not been detected during inspection at the steel works, cannot withstand the twisting process and are rejected. Thus "Isteg" steel comprises separately tested reinforcing bars. The process guarantees a considerable increase in the stress at which co-operation between the concrete and the steel is maintained.

Although "Isteg" steel is new to England it has been used since 1928 on the Continent, where many thousand tons have been employed in concrete structures of all types, including the flooring for the tallest building in Vienna and a bridge in Czecho-Slovakia.

In August 1928 the City of Vienna, after conducting its own tests, permitted

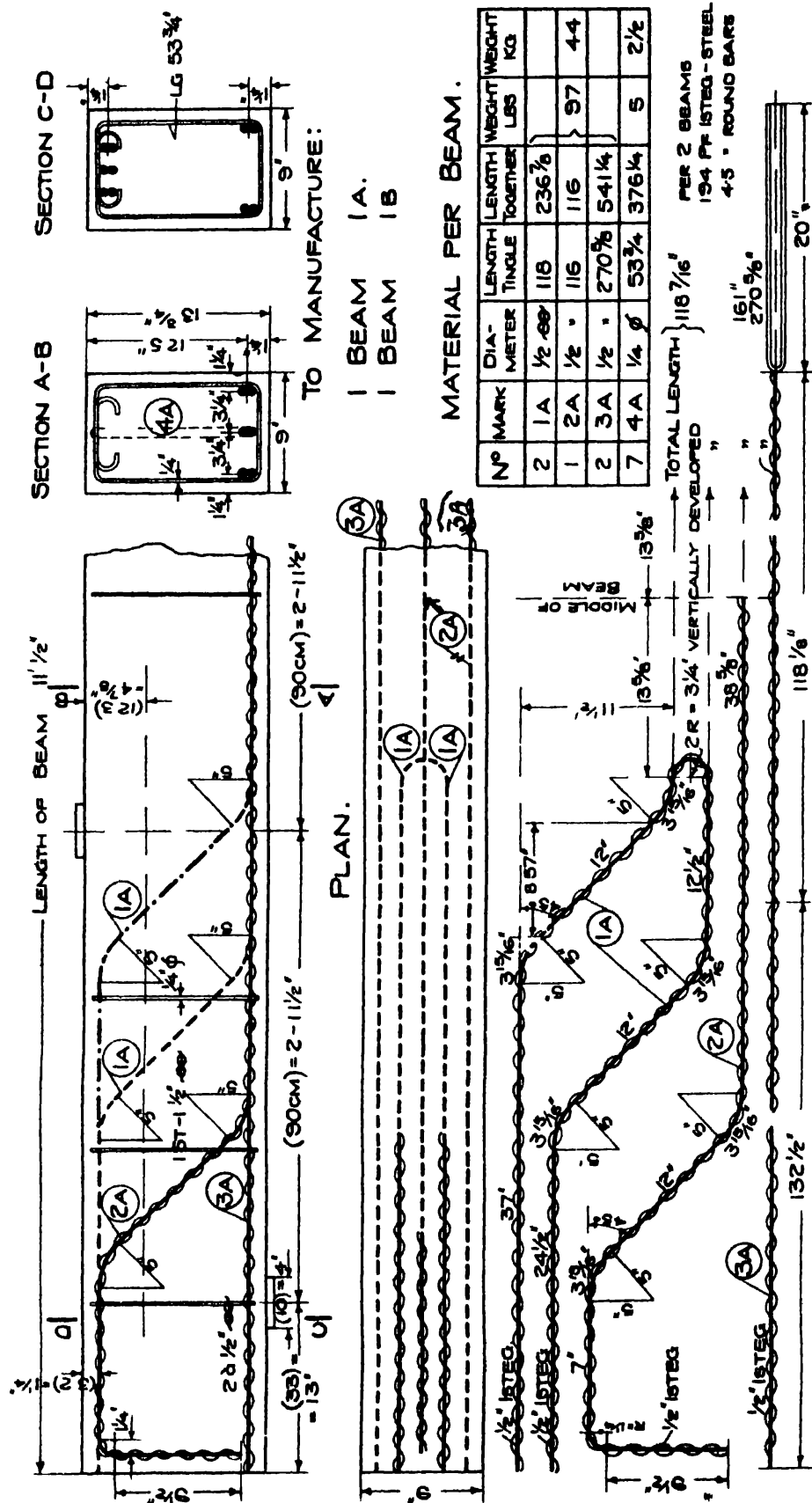


Fig. 4.

" Isteg " steel to be stressed to 24,000 lb. per square inch. The Government of Czecho-Slovakia granted a preliminary decree for 24,000 lb. per square inch, and later, as a result of tests, the decree was amended in 1932 to permit " Isteg " steel to be stressed up to 25,500 lb. per square inch. The State of Prussia in February last, granted a decree for 25,500 lb. per square inch as a result of tests conducted at Dahlem (Berlin) and at the University of Dresden. These figures compare with 17,000 lb. per square inch allowed for ordinary mild steel bars in these three countries.

Tests with British Steel.

Tests were carried out in this country by Mr. R. H. Harry Stanger in May last on four typical reinforced concrete beams, two of which were reinforced with " Isteg " bars and the others with ordinary bars. In each case the steel was ordinary commercial quality supplied by Messrs. Dorman, Long & Co., Ltd., and both sets of bars were from the same cast. Details of a test beam are shown in Fig. 4. In beams Nos. 1 and 3 the reinforcement was $\frac{1}{2}$ -in. " Isteg " steel (0.38 sq. in. cross section), whereas beams Nos. 2 and 4 were reinforced with $\frac{7}{8}$ -in. round bars (0.60 sq. in. cross-sectional area). Each beam was 9 in. wide by 13 $\frac{1}{2}$ in. deep and 11 ft. long, and composed of 1 : 2 : 4 concrete with a water-cement ratio of 0.92 by volume. The aggregates used were $\frac{3}{4}$ -in. Ham River shingle and Ham River sand. Two 6-in. cubes crushed

at 14 days gave a mean compressive strength of 2,009 lb. per square inch. In making the tests the beams were supported on bearers 3 in. wide on a span of 8 ft. 10 $\frac{1}{2}$ in., and the load was applied at the third points of the span. The results of the tests are given in Table I. The beams reinforced with ordinary bars had 58 per cent. more reinforcement than the beams with " Isteg " bars, but the " Isteg " beam withstood the same load and gave a stress of 69,000 lb. per square inch for the steel.

Tests carried out at the University of Prague 18 months ago, using similar beams with the same concrete section reinforced with " Isteg " bars of Czecho-Slovakian open-hearth steel, gave stresses of 68,000 lb. per square inch.

Tests made by Professor S. M. Dixon on two beams 40 days old showed that a saving in concrete is produced by substituting " Isteg " reinforcement for ordinary mild steel bars. In the tests the beams were 11 ft. long, but the section of the beam reinforced with $\frac{1}{2}$ -in. " Isteg " steel was 11 $\frac{1}{2}$ in. by 8 in. while that reinforced with 1-in. plain round bars was 12 in. by 9 $\frac{1}{2}$ in. The cross sectional areas of the concrete in the two beams were 92 and 114 sq. in. respectively, and their weights 1,100 lb. and 1,350 lb. The maximum superimposed loads were 32,700 lb. and 16,860 lb. respectively, the " Isteg " beam, with 24 per cent. less concrete in its cross section, supporting twice the load carried by that containing the plain reinforcement. With the beam

TABLE I

Beam No	1	2	3	4
Reinforcement	$\frac{1}{2}$ -in. " Isteg "	$\frac{7}{8}$ -in. round	$\frac{1}{2}$ -in. " Isteg "	$\frac{7}{8}$ -in. round
Age (days)	34	34	29	29
Weight (lb)	1459	1486	1489	1511
Total load at first microscopic crack in concrete on bottom of beam (tons)	5.5	8.5	7.0	10.0
Total load at which hair cracks were general along bottom of beam (tons)	18.0	14.5	18.0	14.0
Maximum total load (tons)	*21.5	22.5	22.4	22.39
Maximum bending moment (in. tons)	381.6	399.4	397.6	397.4
Maximum shear (tons)	10.75	11.25	11.2	11.2
Maximum tensile stress in steel, assuming modular ratio as 15 (tons per square inch)	29.7	21.3	30.9	21.2

* During the test of beam No. 1 a load of 20 tons was maintained for half an hour. With beam No. 2 a load of 20 tons was maintained for ten minutes only.

NEW FORM OF REINFORCEMENT FOR CONCRETE. **CONCRETE**

containing "Isteg" reinforcement hair cracks did not appear until the superimposed load reached 16,000 lb, whereas hair cracks occurred in the beam reinforced with plain steel at a superimposed load of 12,000 lb. The corresponding deflections at these loads were 0.21 in and 0.14 in respectively. At a load of 16,000 lb cracks developed in the 12-in by 9½ in beam, and in the 11½ in by 8 in beam, at a load of 24,000 lb, a noise was reported to have occurred inside the

beam which was possibly due to failure of the reinforcement

Tensile Tests.

The stress-strain diagram (Fig 1) shows that for "Isteg" steel there is no abrupt yield point as is the case with ordinary mild steel. It is therefore necessary to agree on a definition of the yield point. The German Government in an official decree in February this year,

TABLE II

Results are given below of tensile tests (proposed by Dr Oscar Faber) with "Isteg" steel embedded in concrete (the cross section of the concrete not exceeding ten times that of the steel). Gauge length of tests was 24 in and the concrete section was 2 in by 2 in for ½ in "Isteg" bars and 3 in by 3 in for ¾ in "Isteg" bars. Untwisted tag ends of the original steel were also tested. These test pieces were turned down in lathe. Length of test piece 8 to 10 in.

Test Piece			Yield Point				Ultimate Strength					
Tested Date	Size in	Remarks	Steel Mark	Yield Point taken at fall of the beam		Isteg taken at 0.01 per cent permanent elongation				Remarks		
				Tons per sq in	Lb per sq in	Tons per sq in	Lb per sq in	Per centage of original ultimate stress	Tons per sq in		Lb per sq in	
*1/6/33	1	Tag end of original steel	5	20.5	46,750	75			27.5	62,500	Specimen turned	
*13/6/33	1/2	Tag end of original steel	5	13.5	41,500	20			8	63,100	(check test)	
*8/6/33	1	Isteg embedded	5				26.5	58,000	91	30.3	67,000	Specimen turned
*13/6/33	1/2	Tag end of original steel	5	18.6	41,700	18			27.5	61,000		
*8/6/33	1	Isteg embedded	5				26.5	59,400	96 1/2	30.1	67,400	Specimen turned
*8/6/33	1/2	Tag end of original steel	1	0.5	45,900	67 1/2			30.4	68,100		
*13/6/33	1/2	Tag end of original steel	1	1.3	47,000	7 1/2				29.4	65,800	(check test)
*8/6/33	1/2	"Isteg" embedded	1				27.6	61,400	92 1/2	31.8	71,200	Specimen turned
*8/6/33	1	"Isteg" embedded	1				28	63,100	94	31.6	71,000	
*1/6/33	1	Tag end of original steel	1	18.5	40,000	60			30.4	65,000		
*8/6/33	1	Isteg embedded	1				28.5	63,800	91	32.4	72,000	Works test ultimate 66,600
*8/6/33	1		1				27.1	60,700	81	32.1	71,800	
*8/6/33	1		1				27.0	60,500	91	30.4	68,100	
*8/6/33	1		1									Works tests ultimate 63,200
*8/6/33	1		6				26.1	58,500	92 1/2	31.6	66,300	Works tests ultimate 69,900
*8/6/33	1		6				28.4	63,600	91	33.6	75,300	Works tests ultimate 68,700
*8/6/33	1		6				28.1	62,900	91 1/2	32.8	73,400	Skid machined off test piece
†21/6/33	1	Tag end of original steel	3	0.26	45,380	61				31.97	71,390	
†8/6/33	1	Isteg embedded	3				30.0	67,200	94	35.7	77,800	
†8/6/33	1		3				30.8	68,900		36.3	81,300	
†22/4/33	1		5				27.7	62,000				
†22/4/33	1		5				27.7	62,000				
†22/4/33	1		1				26.5	59,300				
†22/4/33	1		1				28.1	63,000				

* Tested at City and Guilds Engineering College

† Tested by Mr R. H. Harry Stanger

‡ Tested by Federal Testing Station of Laboratory of Technology, Vienna

defined the yield point for "Isteg" steel as being the stress which produces a total elongation of 0.4 per cent. For the purpose of English tests the yield point has been taken at 0.2 per cent. permanent elongation.

If an "Isteg" bar is gripped in a testing machine and pulled in air a certain stress is expended in bringing the two strands of the "Isteg" bar into direct contact throughout the length of the test-piece. In the tests at Berlin and at Dresden University the German Government recognised this difficulty and agreed to take no measurement for elongation in the test-piece until the testing machine registered a stress of 7,200 lb. per square inch and this stress was called the zero point of the test. In England a test suggested by Dr Oscar Faber has been approved in which the "Isteg" bar is embedded in a small section of concrete, it being specified that the concrete section must not exceed ten times the section of the steel test-piece. Results are given in

Table II of a series of these embedded tensile tests carried out by Professor S. M. Dixon at the City and Guilds Engineering College on "Isteg" bars made from steel supplied by five different British steel works; all "Isteg" bars were delivered for test with untwisted tag ends of the original steel. These tag ends were cut off and submitted to the usual tensile tests for yield point, ultimate strength, etc. The "Isteg" bars were then embedded in concrete and after three or four days were broken in a testing machine. It will be noted that the yield points for "Isteg" vary from 58,500 lb. up to 68,900 lb. as against yield points of the original steel (from which the "Isteg" was made) of from 41,700 lb. up to 47,700 lb. If the yield point of the "Isteg" bars is expressed as a percentage of the ultimate strength of the original bars it is found that in only one case did the "Isteg" yield point drop below 90 per cent, the average being well above that figure

Book Review.

"Mitteilungen über Versuche ausgeführt vom Österreichischen Eisenbeton-Ausschuss." Heft 13.

Vienna. Österreichischer Ingenieur und Architekten-Verein. Price \$1.60.

This is the thirteenth report of the Austrian Society of Civil Engineers and Architects dealing with the tests of a committee investigating the properties of reinforced concrete. The contents include a paper by Dr. R. Saliger on experiments on columns containing high-

tensile steel reinforcement, and a report by Dr. Emperger on the strengths of hooped columns. Dr. W. J. Muller contributes an article dealing with the use of pipes made of special concretes, such as the Vianini and the asbesto-cement pipes. The remainder of the volume is composed of a paper on the standardisation of aggregates and an article describing tests of concrete slabs reinforced with expanded metal.

Construction in Winter in Canada.

At the University of Manitoba a new science building of reinforced concrete has been completed without the usual stoppage of construction during the winter.

The building is 230 ft. long with end wings extending backward about 100 ft. It contains four stories and a basement, with a flat roof concealed by a parapet. The building is of reinforced concrete construction throughout, in large part of beam-and-slab type, but in certain parts of slab-and-joist type to make the ceilings flat; the structure rests on a piled foundation. The walls are faced with quarry-faced limestone, laid irregularly and with a pleasing variety of size, surface, texture, colour and jointing.

According to "Engineering and Contract Record" the construction of the building progressed continuously during the winter within an enclosure of $\frac{1}{2}$ -in. insulation board. Within this box, which was warmed to about 45 deg. by air blown over steam coils, operations, including the stone masonry, were carried on without regard to outside temperature or weather conditions

Deterioration of Structures in Sea Water.

IN the latest report * of the committee of the Institution of Civil Engineers which has been investigating the action of sea water on structures of timber, metal, and concrete details are given of the final examinations by Dr J. Newton Friend of the second series of iron and steel specimens which have been exposed at Halifax, Nova Scotia, and Auckland, New Zealand, for ten years. The results so far obtained show that the cast irons, while apparently sound, may have been seriously weakened by internal corrosion, and that the 13.57 per cent chromium steel appears to be quite unsuitable for conditions involving immersion in the sea owing to the severe localised pitting to which it is subject. Of the wrought irons, the Low Moor iron is in general the best in all positions of exposure. The mild steel with low manganese and high sulphur and phosphorus was in some cases in good condition and in others very severely attacked. It thus appears to be an unreliable material. The tests show that the advantages of adding small proportions of copper or nickel to mild steel are less marked after ten years' exposure than after five years. The removal of mill scale before the metal is exposed to the sea is also useful. The wrought irons were cleaned before exposure and should therefore be compared with the cleaned mild steels when the respective merits of the two metals are judged. At both Auckland and Halifax the aerial mild steels in general lost less in weight than the aerial wrought irons, but, in the case of the other two sets, at Auckland the wrought irons as a whole were less corroded than the mild steels, while the reverse was observed at Halifax where corrosion had proceeded to a more advanced stage.

The remaining set of painted steel plates, which had been placed in the open air at Southampton docks in 1924 has now been cleaned and examined. Coatings that proved to be satisfactory include oil paints, particularly the red leads, refined coal-tar, and zinc.

Periodical examinations were made in March and September 1931 of the reinforced concrete test-piles exposed in duplicate, one set in sea water at Sheer-

ness and the other in artificial sea water at the Building Research Station, Watford. At the time of the latest examination the piles had been exposed for periods ranging from 24 to 29 months. Within the last year appreciable deterioration has become evident in a considerable number of specimens, cracking having occurred in several piles having 1 in. of cover over the reinforcement. With the exception of Portland cement-trass mixtures at Watford, all the rich mixtures are free from cracks. The Portland blast-furnace cement mixtures at Sheerness are sound, though cracks have appeared in three of the piles made of lean mixtures of the same cement and exposed at Watford. The disintegration of the lean aluminous cement mixtures is continuing.

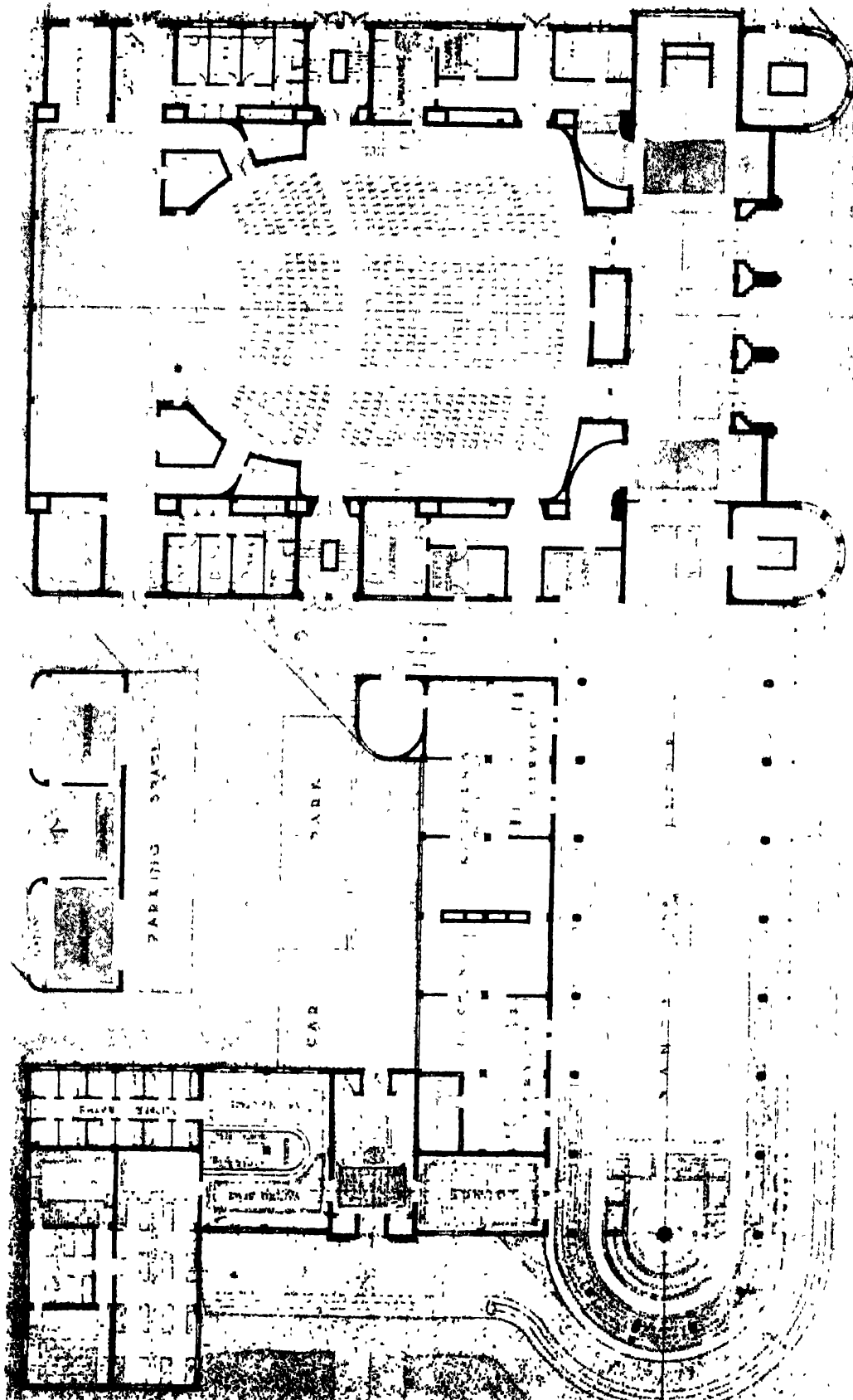
The reinforced-concrete piles exposed to sea action at Sekondi (Gold Coast), which were made of a selection of the mixtures employed for the piles exposed in England and in which the cover over the reinforcement is only 1 in., were examined after having been in position for 24 months. A few rust spots and slight spalling are evident on a number of the piles, while the specimens made of aluminous cement mixtures are somewhat severely cracked at their upper ends.

The successful use of aluminous cement is reported by Mr. W. T. Halcrow, M Inst C E, in a reinforced concrete piled wharf at Kinlochleven. The concrete, which was of 5 : 1 mixture, was placed in 1928, and, though the minimum depth of cover is no more than 1½ in., the work showed no sign of deterioration in July 1931.

Mr. S. H. Ellis, M.Inst.C.E., has furnished particulars of a number of reinforced concrete test-piles that have been partly immersed in sea water at Hong Kong for 11 years, together with information regarding the deterioration of reinforced concrete structures near the mouth of the Prai river. Mr. Ellis's observations point to the conclusion that, in order to obtain adequate protection of the steel reinforcement in a damp tropical climate with good quality 1 : 1½ : 3 concrete, at least 2 in. of cover is necessary. When a richer concrete was employed satisfactory protection was afforded by 1½ in. of cover.

* "Deterioration of Structures in Sea-Water" London. H.M. Stationery Office. Price 1s. 6d. net.

“Concrete and Constructional Engineering” Prize Design.

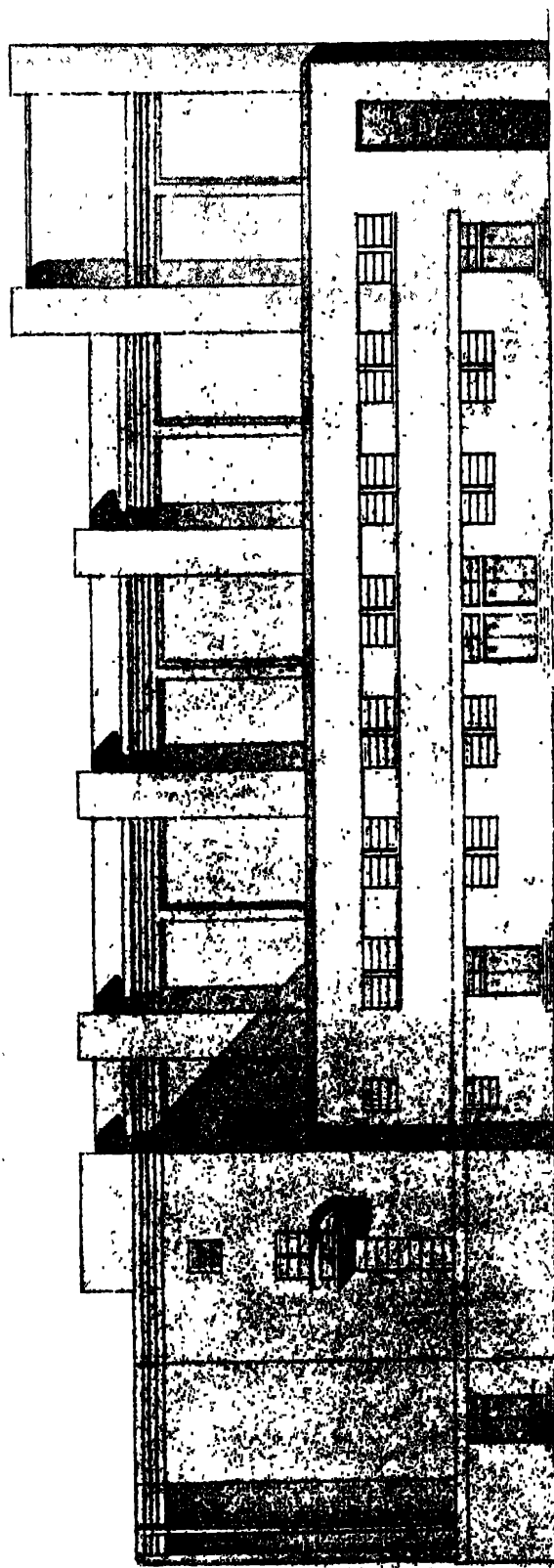


Design for a Seaside Pavilion, Ground Floor Plan. By Mr. E. C. O'Farrell. Awarded First Prize. (See pp. 508, and 542, 544-5.)

“ Concrete and Constructional Engineering ” Prize Design.

PRIZE DESIGN.

CONCRETE

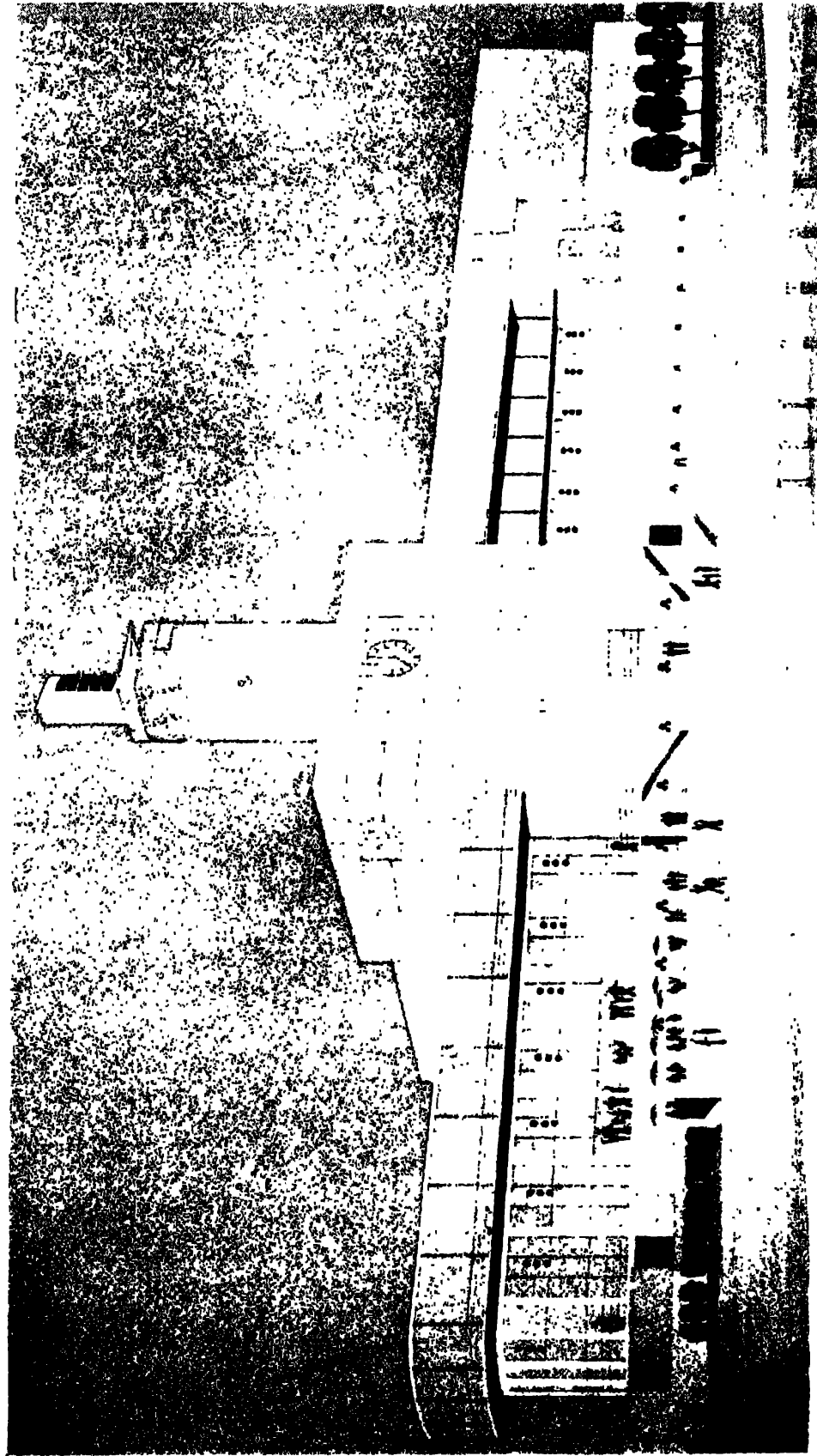


— SIDE ELEVATION TO THE EAST —



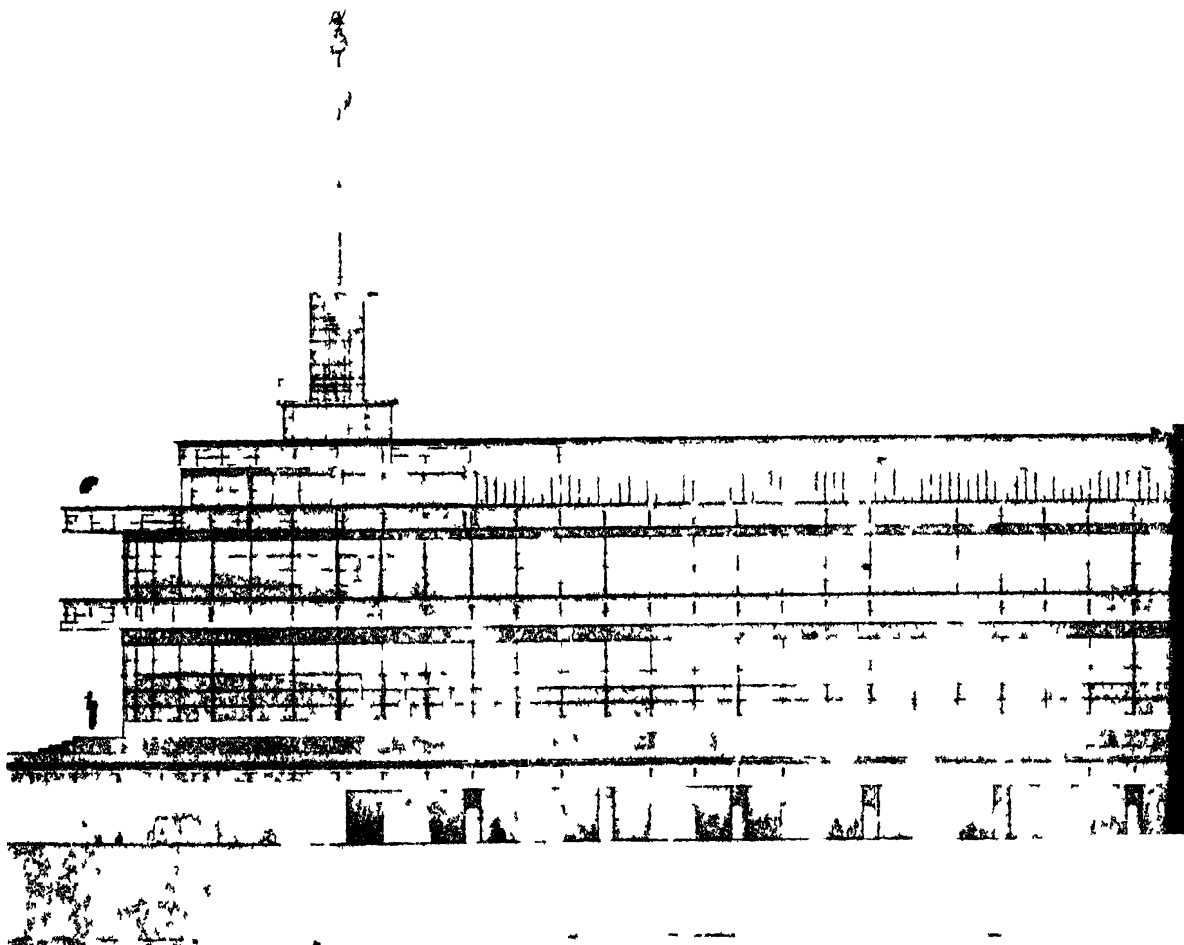
Design for a Seaside Pavillon. By Mr. E. C. O'Farrell. Awarded First Prize. (See pp. 541 and 544-5.)

“Concrete and Constructional Engineering” Prize Design.

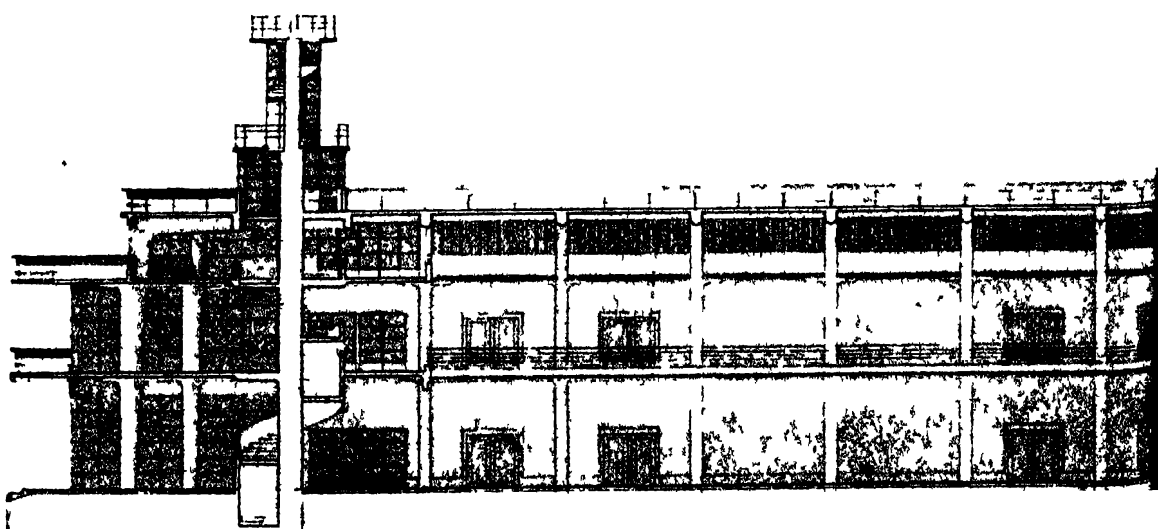


Design for a Seaside Pavilion. By Mr. A. P. Ciregna. (Second Prize.) (See pp. 508 and 546.)

“Concrete and Construction



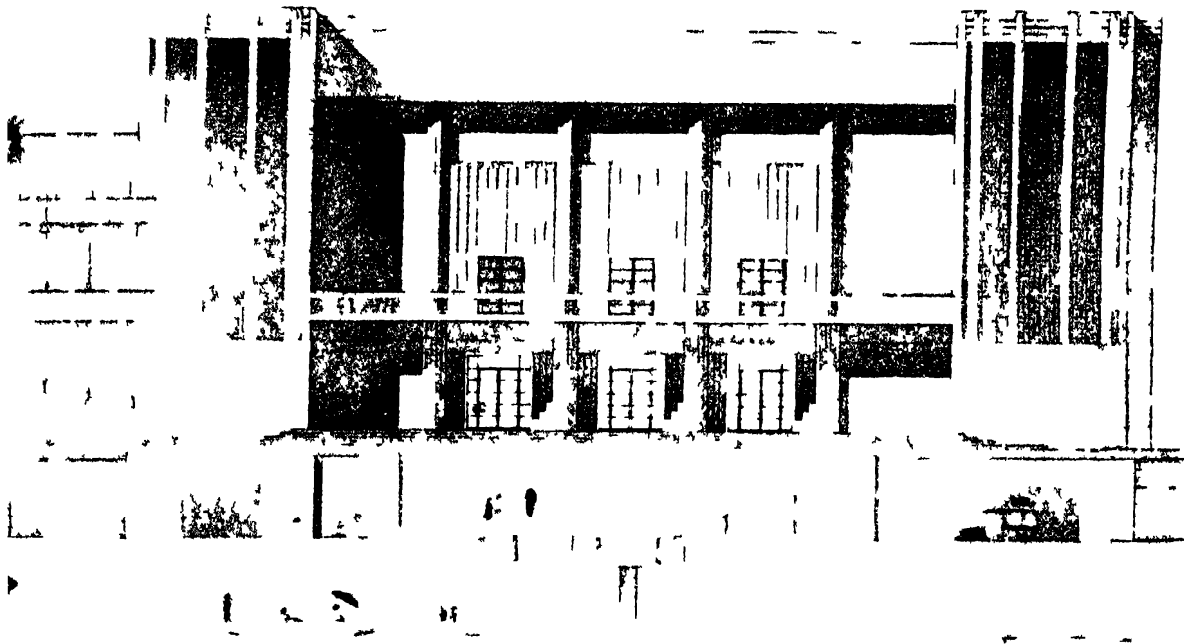
MAIN ELEVATION



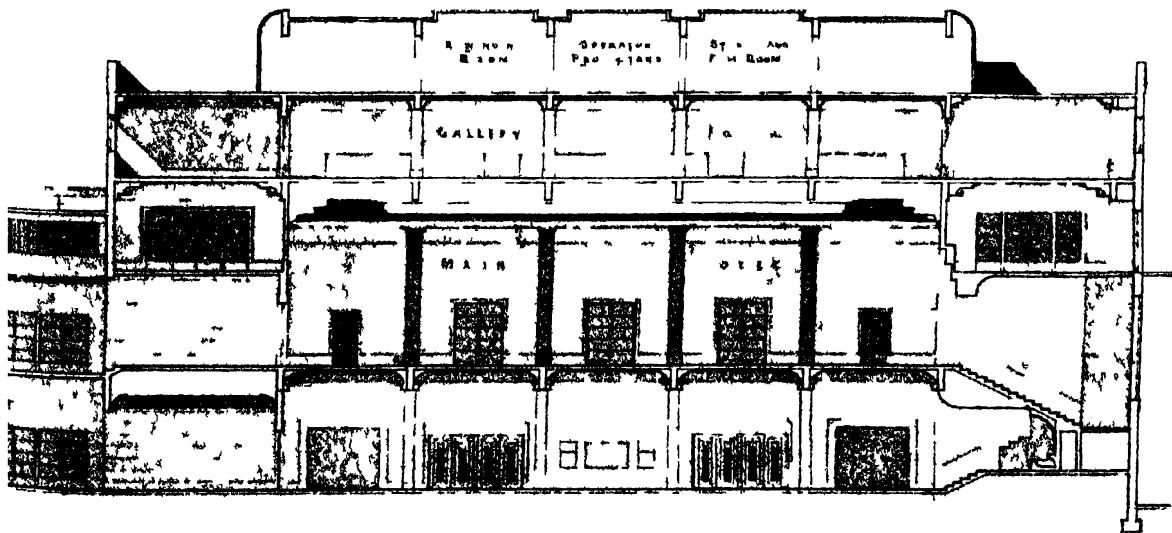
LONG SECTION

Design for a Seaside Pavilion. By Mr. E. C.

Engineering " Prize Design.



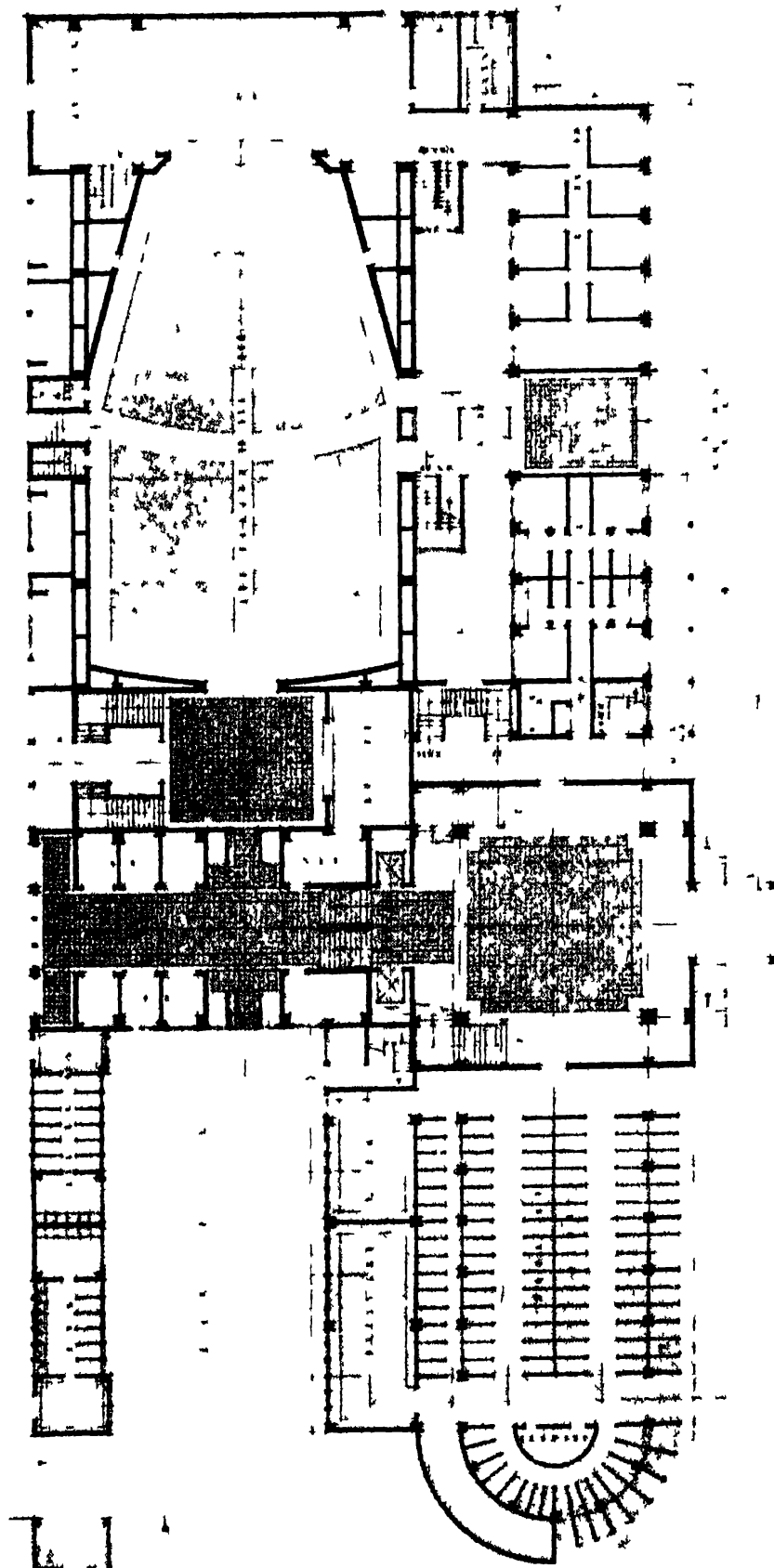
SEA



SECTION

(Awarded First Prize.) (See also pp 541-2)

“Concrete and Constructional Engineering” Prize Design.



Design for a Seaside Pavilion, Ground Floor Plan By Mr A P Ciregna. (Second Prize.) (See also p 543)

The Storstrøm Bridge.

A CONTRACT was signed on May 14 by Messrs Doorman, Long & Co., Ltd., and the Danish State Railways for the construction of a combined rail and road bridge, about two miles in length, across the Storstrøm, a smaller bridge over the Masnedsund which is about one eighth of a mile wide, and about six miles of railway and roads, including the bridge approaches.

These bridges and their approaches will carry the projected railway and road across the two straits, the Storstrøm and Masnedsund which separate the

each end towards the centre at a gradient of 1 in 150, and providing a clear head room for shipping of about 85 ft at the three navigation spans. The centre navigation span is about 450 ft between piers and the two side spans are about 340 ft. The remaining forty seven approach spans are alternately 190 ft and 204 ft between piers. The navigation spans are stiffened tied arches, the arch ribs being of box section type, about 3 ft deep, while the stiffening girders are single web plate girders 12 ft deep over the flange angles. The distance between the girders is about

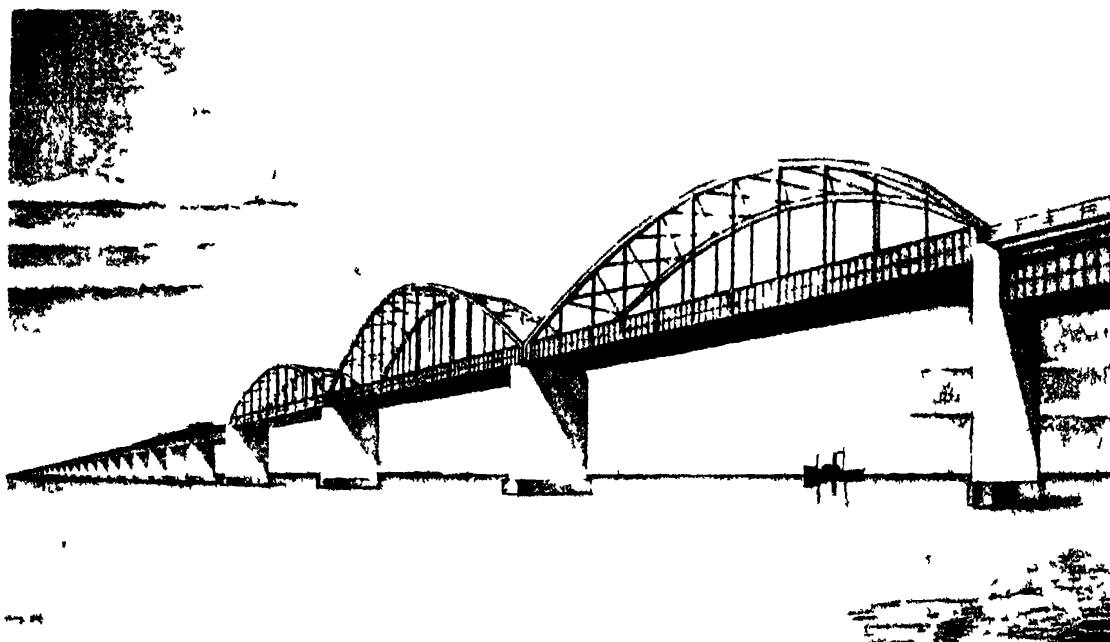


Fig. 1.

islands of Falster, Masnedø, and Zealand thus forming a link in what will be a chain of rail and road communications between Copenhagen on the north-east coast of Zealand with the southernmost point of Falster, leaving only the 25 mile ferry journey between that point and the mainland of Germany to connect Copenhagen with Germany, thus saving about an hour on the journey between capitals. The bridges will carry a roadway approximately 18 ft 6 in wide between kerbs, a footway 8 ft wide, and a single-track railway of 4 ft 8½ in gauge.

The design for the Storstrøm bridge (Fig. 1) contains fifty spans, rising from

10 ft. The hangers are of built up joist section spaced at intervals of 30 ft to 35 ft. Plate cross girders 4 ft deep are framed into the stiffening girders at the hanger points. The railway and road way decks are carried by six lines of joist stringers framed into the cross girders, and the footway is carried on two lines of joist stringers supported by brackets cantilevered from the cross girders.

The roadway and footway are of reinforced concrete slab construction, the roadway being surfaced with 3 in of asphalt and the footway with 1½ in of asphalt. The railway track is carried on

timber transoms with planking to form footways on each side of the track. Lateral wind-bracing systems are provided in the plane of the arch ribs and at the level of the bottom flange of the stiffening girders. The approach spans are of the cantilever type, with suspended spans and anchor arms located in alternate openings, the main girders being plate girders 12 ft deep over the flange angles and 24 ft apart. There are two hinges, situated 30 ft from the supports, in each of the longer spans, so that the whole construction consists of a series of girders with double cantilevers and 150-ft suspended spans between. It is therefore statically determinate. The cross girders are 3 ft deep and spaced at 14-ft 6-in centres, and bear directly on the top flanges of the main girders. Two joist stringers carrying the railway are framed into the cross girders, and four roadway joist stringers rest directly on the top flange. The roadway is of the same construction as on the navigation spans, but the railway track is laid on ballast carried in a reinforced concrete trough and the footway is cantilevered in reinforced concrete from the roadway slab. Lateral wind-bracing is provided in the planes of the top and bottom flanges of the main girders. This bridge when complete will be the longest in Europe.

The Masnedsund bridge design comprises six spans of about 100 ft each, one of which is an opening span of the fixed trunnion-bascule type. The two main girders of each span are plate girders, 8 ft. 4 in deep, spaced about 38 ft apart. The arrangement of cross girders and stringers and the bridge floor are similar to that on the navigation spans of the Storstrom bridge.

Of the 30,000 tons of steel to be used the larger portion will be Messrs. Dorman, Long's new high-tensile steel, Chromador steel, the adoption of which has considerably lowered the cost of the bridge. It will be manufactured in the company's works at Middlesbrough.

The works will contain about 160,000 cb. yd. of concrete and about 2,600,000 cb. yd. of earthworks. Although Messrs. Dorman, Long & Co. are responsible for the whole contract, they will manufacture and erect the steel work only. The remaining portions of the various works will be undertaken by Messrs. Christiani & Nielsen, the agents in Denmark for

Messrs. Dorman, Long & Co., and sub-contractors for the sub-structure and all other concrete construction in the Storstrom and Masnedsund bridges and all works in connection with the bridges. The contract specifies that the whole of the work shall be completed in 4½ years, and the total cost is estimated to be £2,000,000.

The Sub-structure.

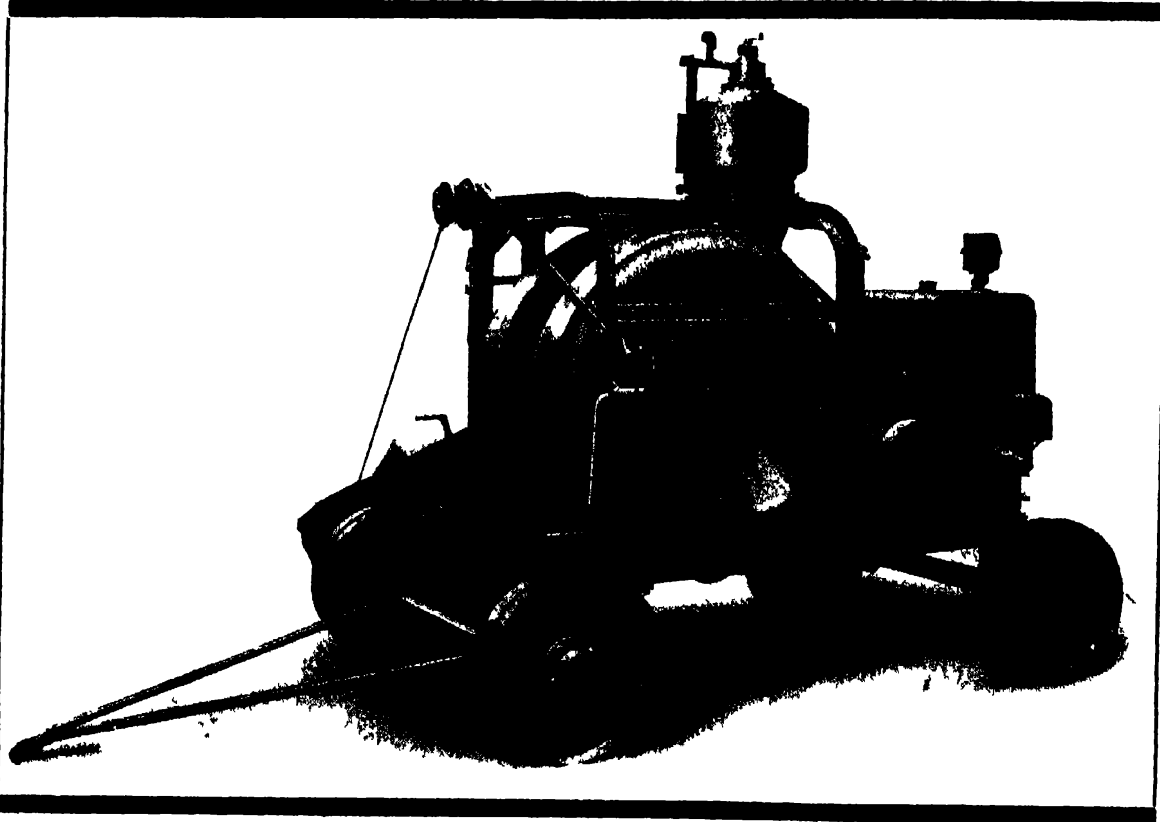
The superstructure is supported on fifty-one piers, all of concrete or reinforced concrete, and the maximum depth of the Storstrom on the line of the bridge is 46 ft. The average depth is approximately 22 ft, and only for a length of approximately 2,000 ft. is the depth more than 30 ft.

On the shore on each side are several layers of mud which have no supporting value, but at a depth of between 20 and 25 ft. there is a layer of clay with a considerable amount of boulders. Limestone rock is found at a depth of 65 ft to 130 ft, but the foundations are not carried down to this depth, as from the borings previously made by the Danish State Railways it is estimated that the supporting power of the clay is sufficient. It is calculated that the greatest depth of the foundation will be 55 ft, while the average depth is approximately 35 ft. to 40 ft. The permissible pressure on the ground has been fixed at 3.2 tons per square foot. Before constructing the piers a number of borings will be made to ascertain the exact depth required for the foundations.

Conditions for the erection of the bridge are comparatively good. Tides are, on an average, not more than approximately 8 in to 13 in., and it is seldom that high water exceeds the level + 3.00. The velocity of the stream is sometimes considerable, but seldom more than 7 ft. per second, and since the sound is sheltered large waves will hardly ever be encountered, but west or south-west gales may cause rough seas. On the other hand conditions with regard to floating pack ice are difficult, as in severe winters floating packs of very great size may form and cause difficulties whilst the piers are under construction. It is therefore important that a method should be used in the construction of the bridge whereby a pier when once started may be speedily completed.



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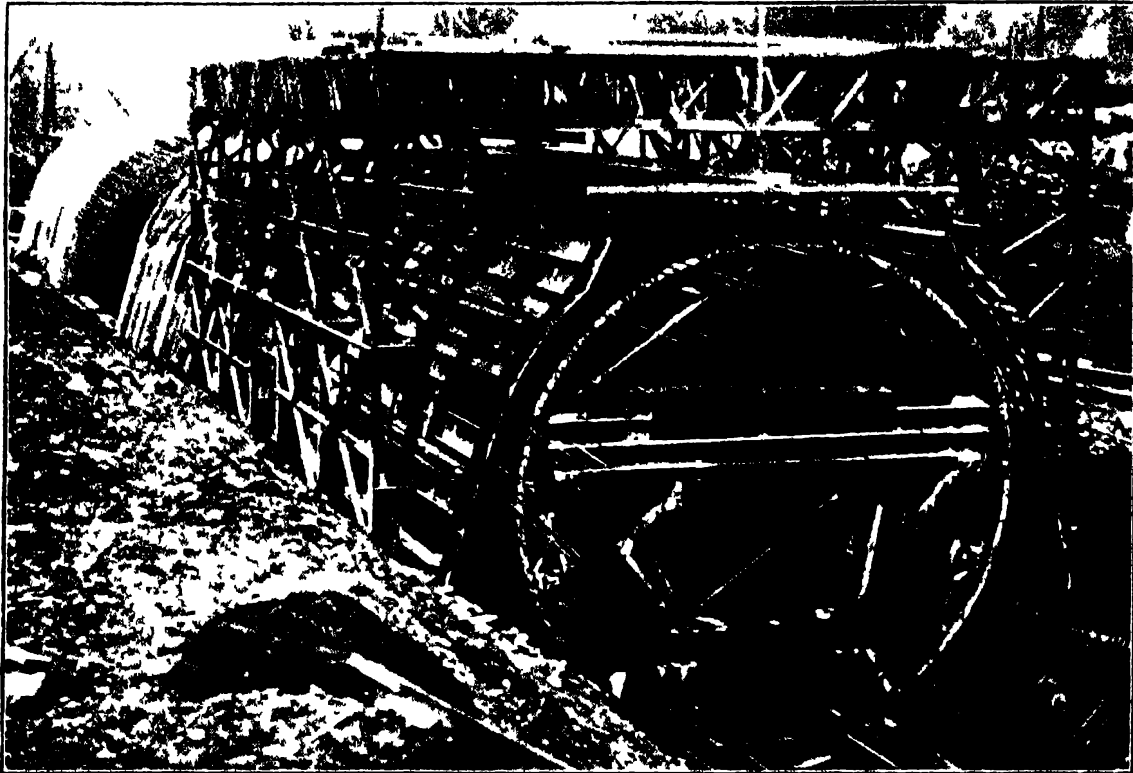
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Pier Construction.

The method of construction of the piers is as follows: Both on Masnedø and Falster the abutments adjoin very high embankments, the tops of which are approximately 60 ft. above water level. The abutments are of reinforced concrete and consist of a system of hollow cells resting on a thick mass-concrete raft. The external, as well as the two secondary longitudinal walls and cross walls, sup-

+ 8.00 the piers will be covered with granite as a protection against water and floating ice. Above level + 8.00 the piers will be of cellular construction. Around all the foundations steel sheet piling is to be driven to 10 ft. to 15 ft. below the foundation raft, and will act as a safeguard against undermining of the piers by the current. The possibility of further protection by stone filling has also been taken into consideration

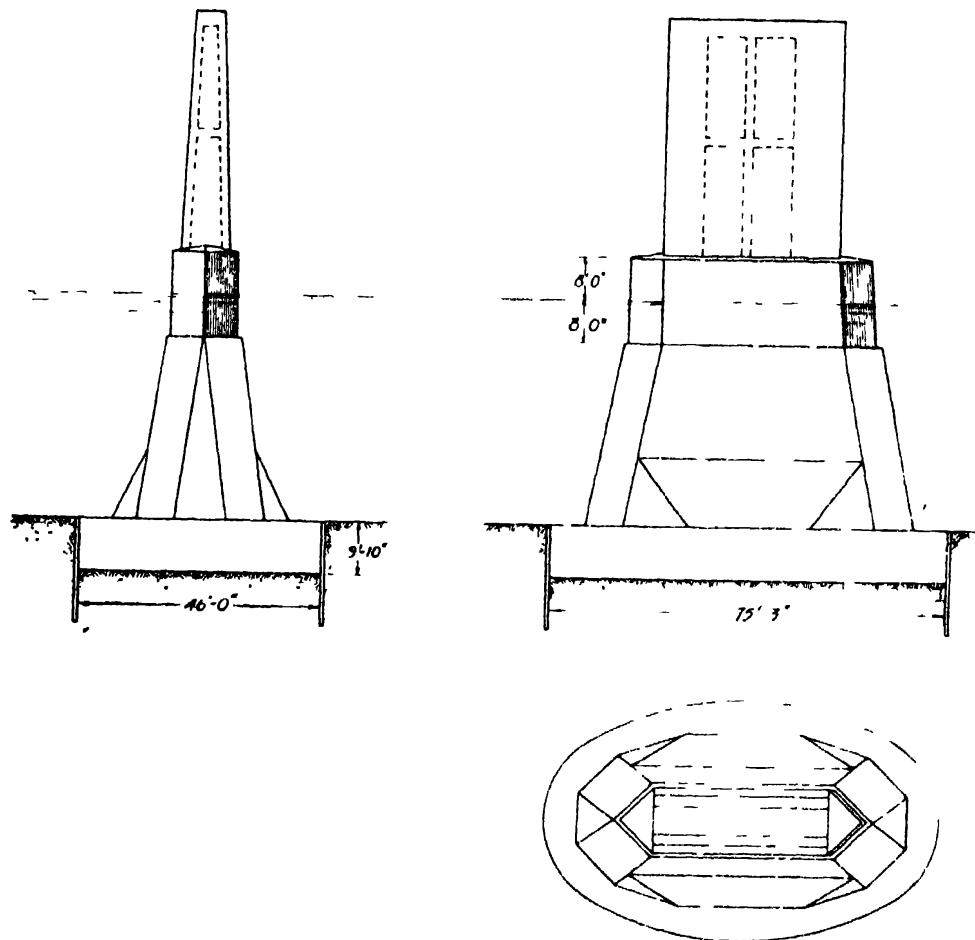


Fig. 2.

port the bridge superstructure which is cantilevered back so that the distribution of the load on the foundations is approximately uniform. These abutments will be made within a cofferdam of steel sheet piling which will remain in the building as a protection against undercurrents

The forty-nine intermediate piers are to be made of mass concrete, the foundations consisting, on the average, of a 10-ft. thick concrete slab, elliptical on plan. At the water line, from the level - 8.00 to

The great number of intermediate piers standing in nearly the same depth of water and subject to practically the same forces makes it possible to standardise the execution of this part of the work, and the piers are divided into a few standard types. The part of the pier which is under the water can be made the same for all piers of the same type, and the same shuttering can be used for all. The four piers at the navigation openings in the centre of the bridge, where the con-

struction of the superstructure is different, are necessarily of considerably greater dimensions than the others. Some piers standing in shallow water are smaller and not suitable to be made as standard types. Those at the centre openings and in the shallow water will be constructed in an open excavation inside a cofferdam of steel sheet piling. When the piers are built up to a level just above the water, the steel sheet piles will be cut off under water at the level of the top of the foundation raft.

The remaining piers, approximately forty in number, will be built with the aid of special standard units. Each of these is a floating body built of steel, the base being elliptical on plan externally, corresponding in size to the elliptical pier base, internally these structures will be made to act as shuttering for the lower part of the pier shaft. These units will first be used as staging for the driving of the steel sheet walls round the foundation. The steel sheet walls are only to be driven down to the length in which they will be required in the finished construction, and the units are sufficiently strong to serve as strutting for the steel sheet walls and as cofferdams for the pier shaft when the water in the units is pumped out after the sheet piles have been driven and tightened against the units.

The units also support the machinery necessary for keeping the pit dry, for excavating for foundation, etc., and the shuttering for the top of the piers, whilst they act as centering for the lower part of the piers. The units are also fitted with water tanks by means of which they can be raised or lowered by pumping water out or in.

When a pier has been constructed to a certain height above the water level the unit will be removed and the top part of the pier will be constructed in the ordinary way. As an example of the construction of a pier which will be built by means of the unit described, the illustration (*Fig. 2*) gives the main dimensions of the largest standard type.

For the sub-structure of the Storstrøm bridge 12,000 cb yd of concrete will be required, whilst for the bridge slab, which is in reinforced concrete, 12,000 cb yd. will be used. It will be necessary to erect large floating plants to place this quantity of concrete.

The comparatively short time of 4½ years for the building of all the bridge work, including the time required for the more detailed examination of the stream bed, will make it necessary when the work is in progress to finish the lower part of a pier in approximately seven weeks.

Recent Patent Applications.

- | | |
|--|--|
| 393,429.—Holzmann Akt.-Ges. Zweigniederlassung Berlin: Manufacture of light concrete | 393,864.—C. F. Ball: Concrete mixing and agitating apparatus |
| 393,576.—E. Freyssinet: Process and apparatus for manufacturing reinforced concrete sections | 394,505.—L. M. Fairclough: Concrete flooring. |
| 393,605.—G. Jaeger, and J. Eggert: Device for supplying water to concrete mixers. | 395,256.—H. Camus: Shuttering for concrete. |
| 393,641.—Garvenswerke Maschinenpumpen und Waagen-Fabrik W. Garvens: Apparatus for the production of concrete piles | 395,368.—G. Carpenter: Construction of walls, ceilings and partitions. |
| | 395,442.—A. Grant: Hinge for obtaining flexibility of formwork for concrete construction |

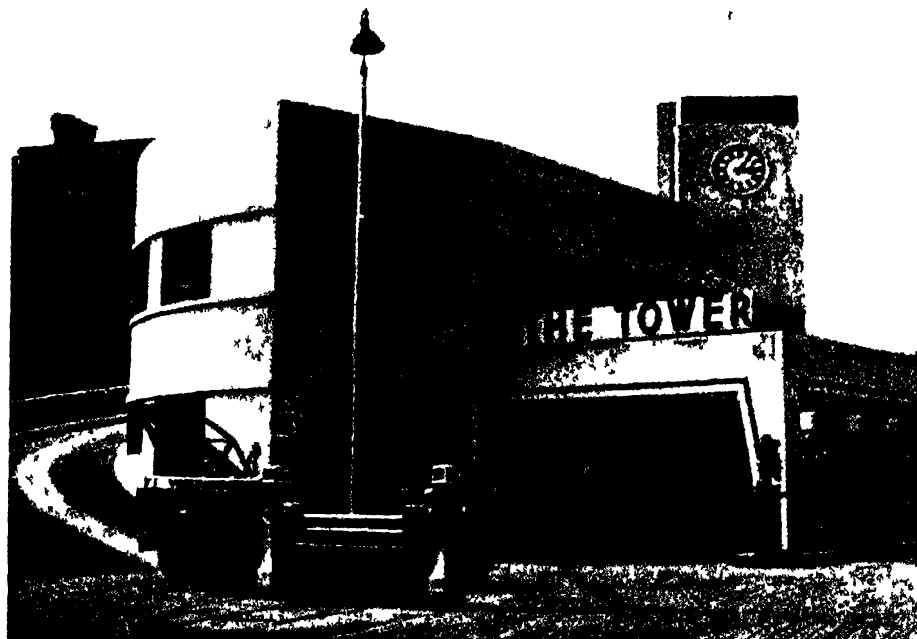
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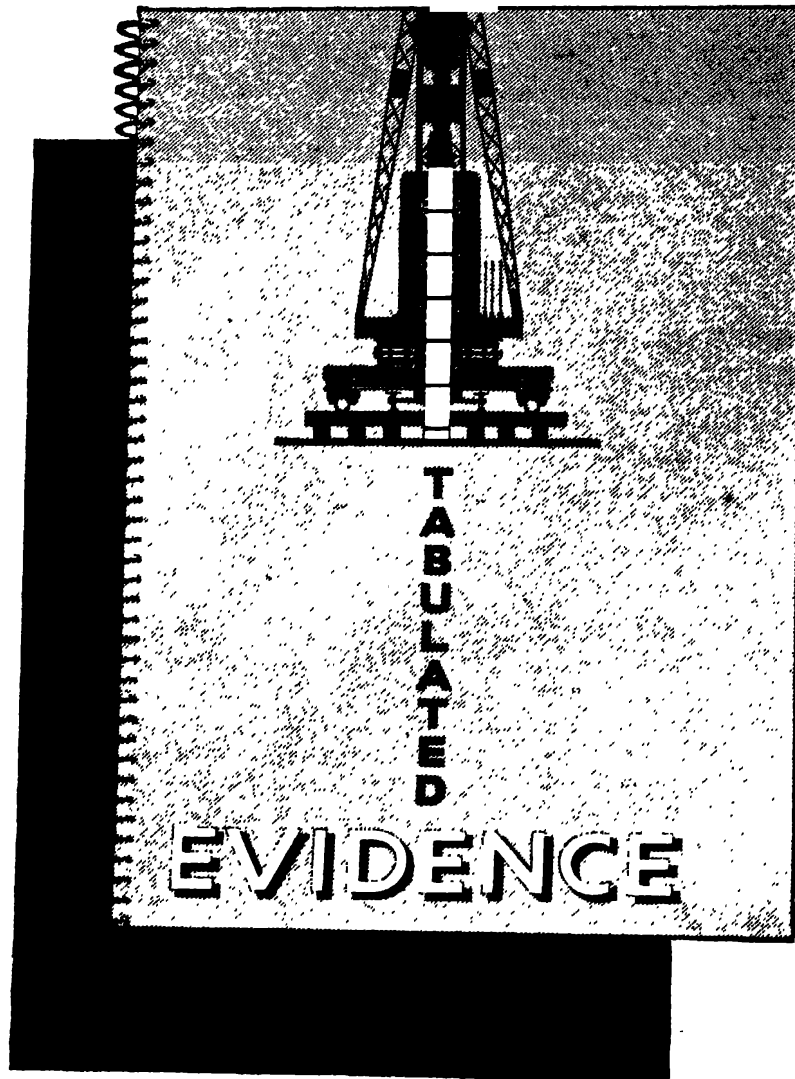


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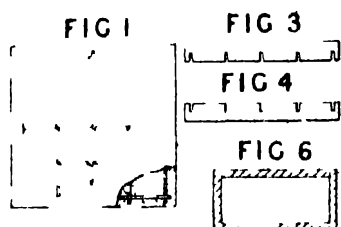
This new booklet gives tabulated data of many interesting contracts carried out by West's Rotinoff Piling & Construction Co., Ltd. (Regent House, Kingsway, W.C.2) ; a copy will be sent on request to anyone interested in piling.

Recent Patents Relating to Concrete.

Reinforced Blocks, Bricks, etc.

369,241.—Berry A F Wellington House Strand London Oct 10 1930

Blocks, slabs, bricks etc of concrete rendered cellular by internal generation of gas are strengthened by reinforcements which extend through or around substantially their whole length breadth and thickness. The reinforcement may be internal for example interlocking metal strips as in *Figs 1 3 4* or external for example a baked clay container with a lid (*Fig 6*) or both. Other materials such as asbestos concrete may be used for the reinforcement which may take other forms (e.g. hollow wavy angular). The concrete consists of Portland or other cement mixed with broken burnt clay or asbestos etc and a small quantity of finely divided alloy or metal (e.g. aluminium with optionally a chemical agent such as sodium hyposulphite). The concrete is wetted, picked into or around



REINFORCED BLOCKS BRICKS ETC

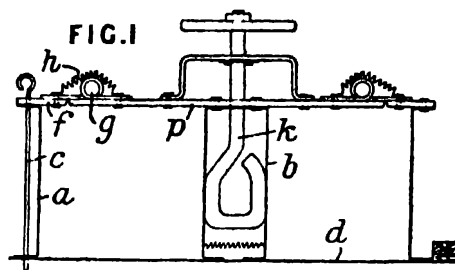
the reinforcement and allowed to rest preferably in a warm state while gas is generated. The unset blocks may be enclosed in a chamber and subjected to (1) reduced pressure to promote coarseness of cell (2) warm air for drying (3) a fluid hardening medium (e.g. a solution of magnesium silico fluoride or sodium silicate). The blocks may be covered with cement plaster paint etc veneer leather or fabric.

Casting Walls in Situ.

369,285.—Schmidt J T Nebel on Amrum Germany June 8, 1931

Concrete walls are cast course upon course between movable metal moulds which are raised immediately after the casting of one course and are supported on that course. The courses are formed of layers not more than 25 cms in height and the aggregate used is only slightly

moistened. The outer moulds comprise flanged metal plates (*a*) supported on cross bars (*d*) on the previously cast course and upper distancing members (*p*) are used on the moulds pins (*c*) passing



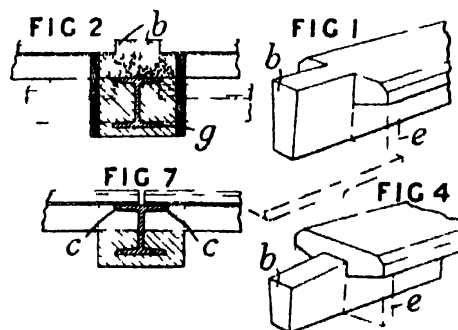
CASTING WALLS IN SITU

through the ends of these distancing members and through apertures in the flanges of the moulds. The end parts (*f*) of the upper distancing members are secured to the moulds and are hinged to the main part (*p*) and springs (*h*) are used to pull these parts upward and so release the moulds from the cast surfaces as soon as the pins (*c*) are withdrawn. For hollow walls core plates (*b*) are fixed to the upper distancing members and these are expanded by a rotatable key (*k*) and are spring retracted.

Pre-cast Beams for Floors, etc.

370,164 Morton B and Morton H D Edgeworth Hawley Inc Hale Cheshire March 17 1931

Floor or roof structures composed of pre cast concrete beams of T section in which the upper webs form the entire



PRE CAST BEAMS FOR FLOORS ETC

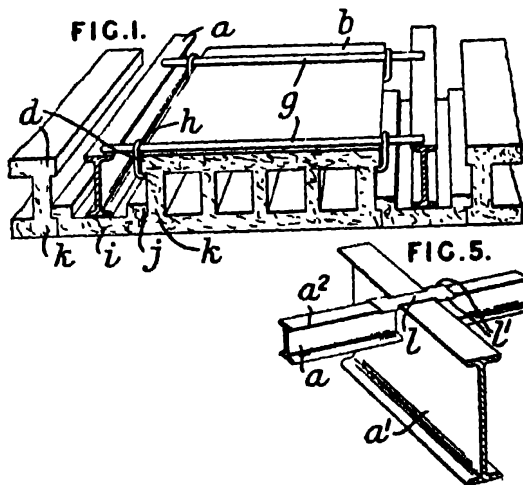
floor surface have at each end a web like extension formed by omitting parts of the top flanges or by slotting the beam from each end, the beams resting by

their extensions on supporting girders or their haunching and being fixed in position by grouting filled into the spaces between the extensions. *Fig. 1* shows a form in which the extension (*b*) is formed of the whole depth of the web. In *Fig. 4* the extension (*b*) is of less depth and has a sloping upper surface; the upper surface may also be horizontal. *Fig. 2* shows the beams resting on a girder (*g*) and fixed by grouting. The beams may be formed with lateral flanges (*c*, *Figs. 2* and *4*) which will avoid the need for shuttering for retaining the grouting, or pre cast blocks cemented in position may be used instead of grouting. *Fig. 7* shows a form in which the web is slotted at (*c*), the beam being supported on haunching. The upper surface may be flat or slightly arched, and the edges may be furnished with inter-engaging tongues and grooves. The undersides may be covered by expanded metal, etc., to form a ceiling.

Floors.

371,978.—W. G. Shipwright, 102, Abbey House, Victoria Street, London Feb. 11, 1931.

Hollow tiles (*b*, *Fig. 1*) are supported from joists (*a*) by metal rods (*g*) having members (*h*) which engage under the top flanges (*d*) of the tiles (*b*). In a modification, the rods (*g*) are replaced by metal



FLOORS.

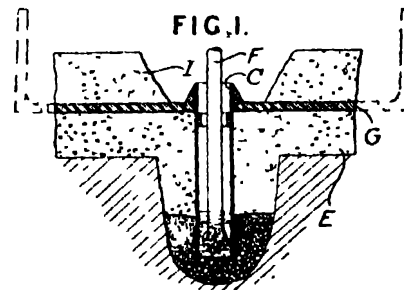
plates having tongues which engage under the flanges (*d*) of the tiles (*b*). Further tiles (*i*) may be employed to fill the spaces between the tiles (*b*), these tiles (*i*) being formed with flanges (*j*), which rest on bottom flanges (*k*) of the tiles (*b*). The

joists (*a*) are supported from main beams (*a*¹) by metal or other plates (*l*, *Fig. 5*), which rest upon the main beams (*a*¹) and have their ends (*l*¹) bent to engage under the top flanges (*a*²) of the joists (*a*), or the plates (*l*) may be replaced by rods formed with hooks which engage under the top flanges (*d*) of the joists (*a*).

Waterproof Floors.

372,709.—T. G. Marriott, Artillery House, Artillery Row, Victoria Street, London. Jan. 9, 1932.

Waterproof floors and linings for chambers below water level are formed by laying first a concrete or like bottom layer (*E*) with pipes (*C*) extending through into the subsoil, so that water may be removed during construction by means of a hose (*F*)



WATERPROOF FLOORS.

inserted into the pipes. An asphalt or similar dampcourse (*G*) is then laid and sealed around the pipes, and the top layer (*I*) of concrete is so laid as to leave temporary gaps to give access to the pipes. When no more water is to be removed the pipes are closed by plugs and sealed with an asphalt filling, and the top layer (*I*) is completed by filling in the gaps, the pipes being left in situ.

Treating Slag ; Artificial Stone.

374,008.—Dunn, E. P., Ferny Creek, Victoria, Australia. Feb. 23, 1931.

A process for treating molten slag to produce constructional material, aggregates, artificial stone, etc., of uniform composition, consists in removing impurities from the slag by settling and also by oxidation, adding substances, e.g. silica, to combine chemically with the slag, and mechanically incorporating solid materials. The preliminary step of agitating the slag to homogenise it is preferably included. Apparatus shown in *Fig. 1* includes a settling device and



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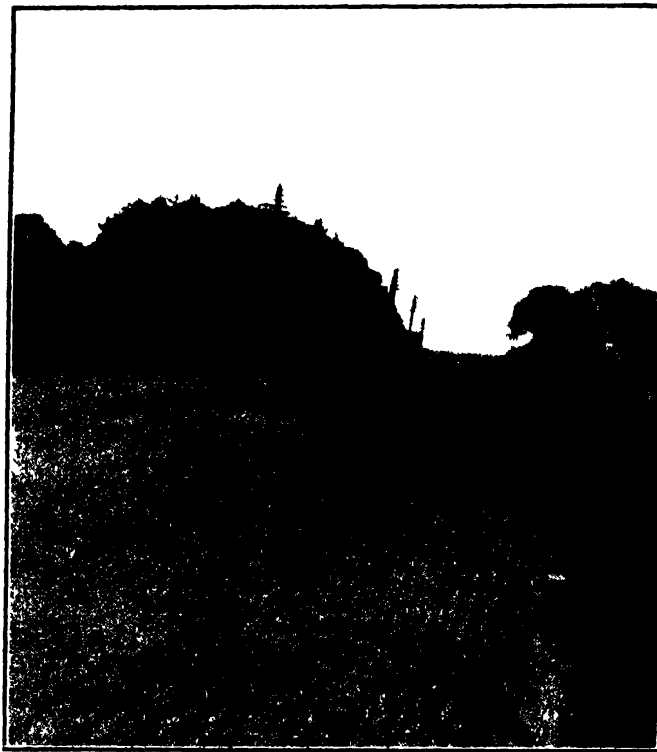
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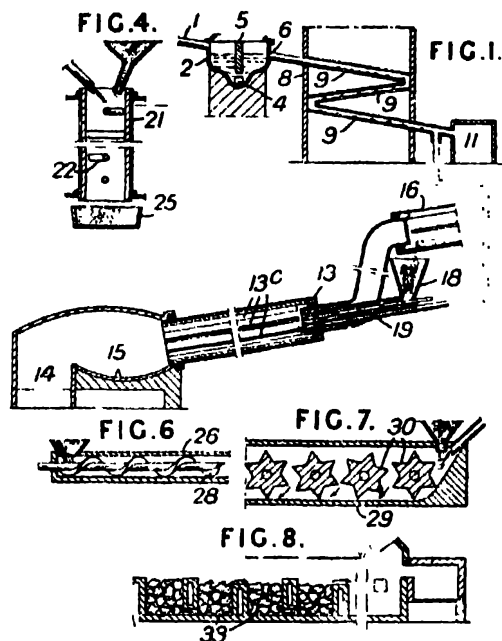
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mixer (2), a desulphurising furnace (8) and a rotary furnace (13). The slag flows into the mixer (2) down a chute (1) and therein passes under a baffle (5) to the outlet (6). Solid particles settle and are removed from an outlet (4). In the desulphurising furnace the slag flows in



TREATING SLAG

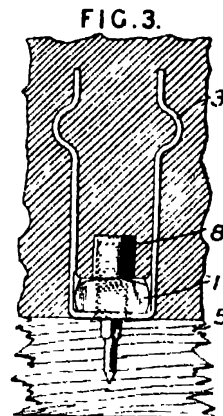
a thin stream over a series of inclined beds (9) where it is exposed to oxidising gases from a firebox (11). The beds (9) are formed with projecting ridges or teeth to break up the stream. The slag then passes to a rotary furnace (13) in which material such as sand is chemically combined with or dissolved in the slag. The sand is preheated in a rotary drum (16), heated by the waste gases from the furnace, and from the drum passes to a hopper (18) and is charged into the furnace by a screw conveyer (19). The furnace is formed with internal ribs (13c) adapted to lift and drop the material. When the material to be combined is in lump form, a furnace shown in *Fig. 8* is used instead of the rotary furnace. The lump material is placed in a series of compartments (33) through which the slag flows in succession. Solid materials may be incorporated with the molten slag in a rotary vertical cylinder (21, *Fig. 4*), having transverse bars (22) which may be hollow and cooled from within. The slag and added material are fed into the top of the cylinder and the mixed

composition falls into moulds (25). Alternative mixing devices are (a) *Fig. 7*, a narrow trough (29) containing a series of star wheels (30) which may be hollow and cooled; (b) *Fig. 6*, a fixed cylinder (26) containing a revolving mixing screw (28); and (c) a modification of the device shown in *Fig. 6* having the screw fixed and the cylinder revolving.

Blocks with Inserts for Attaching Fittings.

374,536.—H. P. Ford, 57, North Road, West Bridgford, Nottinghamshire March 18, 1931.

Sockets, such as nuts (1), forming means for attaching fittings, are secured in a cast mass, for example a ceiling, wall, or floor, by fixing them temporarily by means of pointed studs (5) driven into the shuttering and then casting the concrete around them. In the case of a nut, the stud is screwed at the upper end for subsequent removal from the finished article and a cap or tube (8) of fibre, paper, or the like may be used to cover the projecting upper end and prevent the concrete binding in the threads. The nut



BLOCKS WITH INSERTS.

may be surrounded by, and in some cases welded at the sides to, a stirrup (3) curved towards the upper end to embrace one or more reinforcing rods.

Reinforcing Fabric.

374,582.—P. S. Reid, Newlands, Hill-side Road, Ewell. April 30, 1931.

In mesh reinforcement the longitudinal members are interconnected by links enabling the fabric to be folded in concertina fashion. The links may be formed at their ends to clip on to the longitudinal

members. In the form shown, the links (b) span from one bar to an adjacent bar, and comprise V-shaped links made from a continuous length of wire and turned up at the ends of the limbs to form clips (d), and at the junction to form corresponding

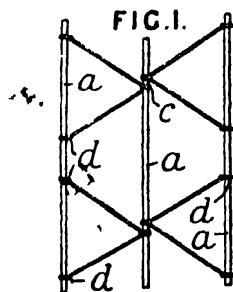
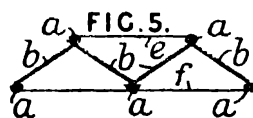


FIG. 2.
d c

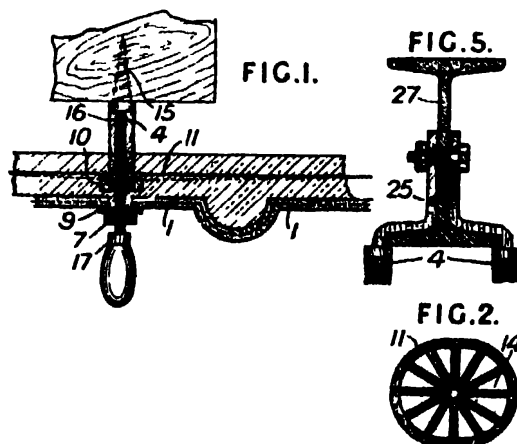


REINFORCING FABRIC

clips (c). Where some of the bars (a) alternate with others at a different level, additional links (e, f) may be used to connect the bars on the same level. Instead of being V-shape the links may be straight or of any other form and may be turned up to form clips of different shape. Radially arranged bars for circular reinforcements are connected by links of different lengths.

Moulding Ceilings In Situ.

376,065.—W. Süßmilch, 40, Grassi-strasse, Leipsic, Germany. Aug. 21, 1931.



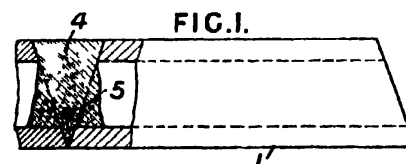
MOULDING CEILINGS IN SITU.

Supports for ceiling shuttering are provided with means for attachment to the floor joists and with axial bores to receive screws having means to support the shuttering. The supports (4) are provided with wooden screw or spiked ends (15) for attachment to wooden joists and have a bore (16) for a screw (17) provided with an adjustable nut and washer (7 and 9) on which rests the shuttering (1), which may be plain or ribbed and formed of reinforced glass. Metal reinforcement for the ceiling may be formed of an open dished metal disc (11) the ribs (14) of which may be corrugated, and this reinforcement is supported on a nut (10) carried by the support (4). For attachment to metal beams (27, Fig. 5) the cranked ends (25) of the supports (4) are perforated to receive a bolt.

Hollow Blocks for Concrete Floors.

376,152.—R. Stransky, 12, Mariahilferstrasse, Vienna. Nov. 12, 1931.

A hollow block (1) of the kind having longitudinal lips at its lower edges which abut with the lips of adjacent blocks to



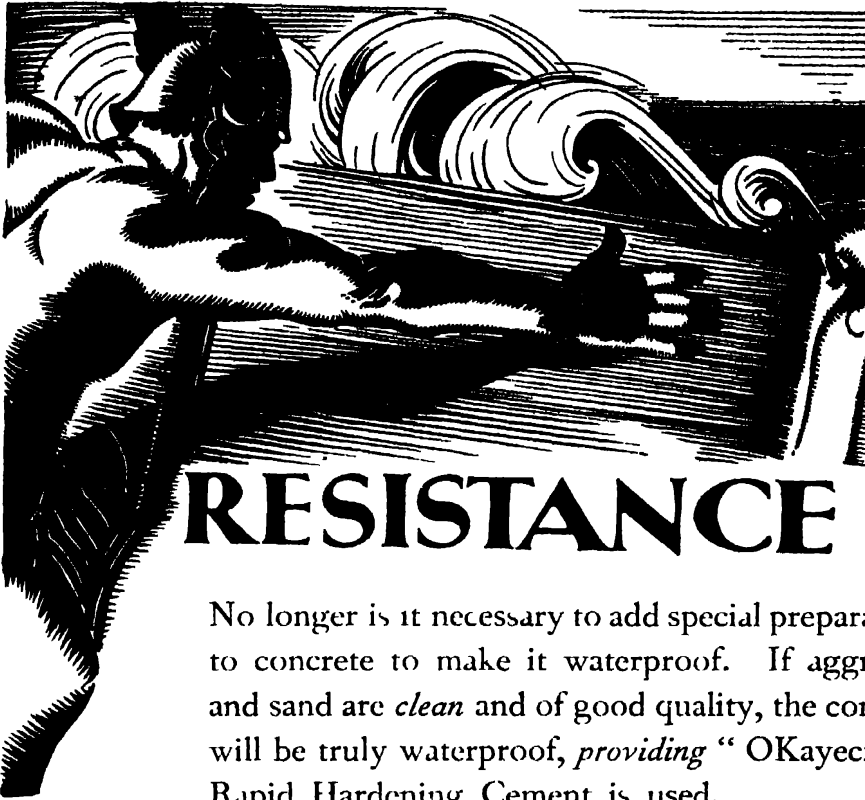
HOLLOW FLOOR BLOCKS

form moulds for concrete joists has its ends chamfered to the full depth of the block to form a transverse V-shaped space (4) into which the concrete enters and thus partly enters the blocks and keys them together. Transverse reinforcing rods (5) are provided.

Floors.

376,228.—L. Navratil, 15, Vilmos császár ut, Budapest. Dec. 30, 1930. Void.

In reinforced concrete floors of the kind in which hollow bodies are disposed between reinforced concreted carriers two or more adjoining hollow bodies are arranged in series at right-angles to such carriers between adjacent carriers. As shown in Fig. 1, two adjoining hollow bodies (2a, 2b) are arranged between adjacent reinforced concrete carriers (1), the contact walls of the bodies and carriers diverg-



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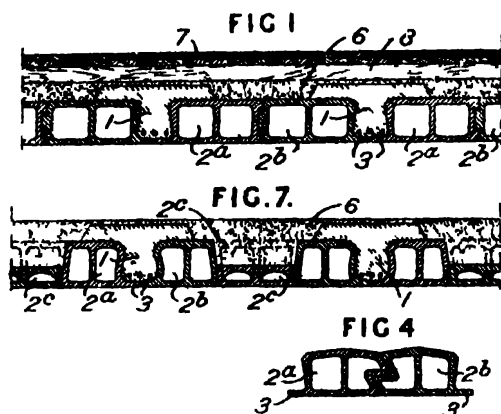
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ing downwardly and the contacting faces of the hollow bodies being provided with tongues and grooves. The hollow bodies may be interlocked in various ways or may be cemented together. Preferably flanges (3) are provided. Fig 7 shows a floor in which three adjoining bodies (2a



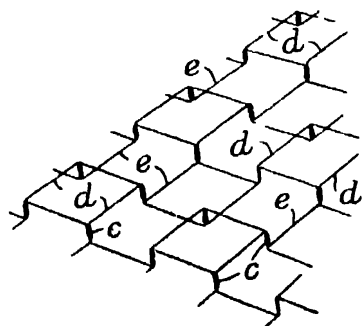
FLOORS

2d, 2e) are provided the intermediate body (2c) being of full or diminished height and having wedge shaped walls adapted to key with the adjoining walls of the bodies (2a, 2b). Slag etc filling (6) is provided upon which are superimposed the floor beams (8) and floor boards (7).

Reinforced Concrete

376,580. P. S. Reid, Newlands Hill, side Road, Fwell, Oct 16, 1931.

A reinforcement particularly a spacing reinforcement for road construction com-



REINFORCED CONCRETE

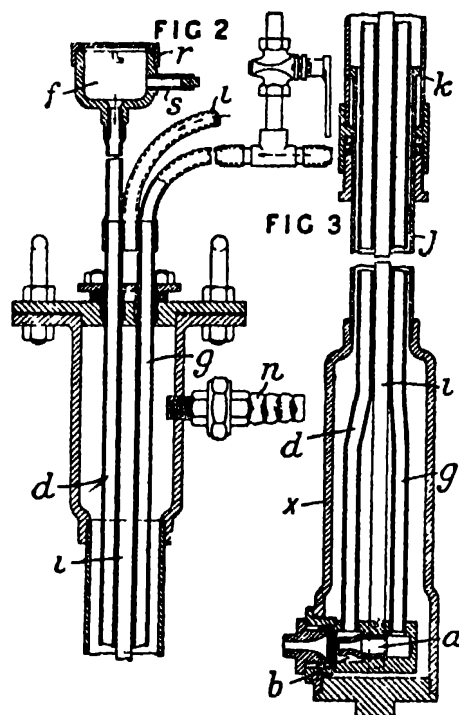
prises continuous lengths of wire etc having upper and lower parallel portions (d, e) and interconnecting vertical parts (c). The lengths are assembled as shown with the vertical parts (c) bound by wire,

thus enabling the assemblage to be collapsed for transport. The reinforcement may be used for constructing walls.

Cement Guns.

382,425.—West, F. J. West, L., and West's Gas Improvement Co. Ltd, Albion Ironworks, Miles Platting, Manchester, Oct 28, 1931.

Vertical retorts are repaired by spraying a liquid medium such as silica clay grouting on to the defective part by an air blast from a nozzle, the conduits to



CEMENT GUNS

which are telescopic and are cooled to prevent drying of the repairing medium and to provide a cold air blast for use after spraying to prevent premature setting of the repair. The spraying head (r) contains an air nozzle (a) and an annular space (b) to which the repair liquid is supplied through a pipe (d) from a container (f), the air being fed through a pipe (g). Telescopic pipes (j, k) enclose the air and liquid pipes and are supplied with cooling water from a pipe (i), an overflow (n) being provided. The repairing liquid may be supplied under pressure by closing the container (f) with a cap (r) and connecting a pipe (s) to a pressure air supply.

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Best Washed Sand	per yard	7	9
Clean Shingle, $\frac{3}{4}$ in. mesh	"	6	9
" " $\frac{1}{2}$ in. mesh	"	9	0
Thames ballast	"	6	9
Broken brick ($\frac{3}{4}$ in.)	"	10	6
Best British Portland Cement (delivered London area) per ton 46s., including non-returnable paper bags; 44s. 9d., including charge for hire of jute sacks.			
Rapid-Hardening Portland Cement	delivered London	7s.	6d. per ton extra.
" Colorcrete " red and buff rapid-hardening Portland Cement,	delivered London	20s.	per ton extra

"Snowcrete" White Portland Cement, £9 10s per ton delivered London, including non-returnable paper bags.

"Super-Cement" Waterproof Cement (including paper bags) per ton 76s.

BOARDING FOR SHUTTERING—		Sawn.	Wrot.
1 in.	per square	22s. 6d.	26s. 6d.
1½ in.	"	28s. 6d.	32s. 6d.
1½ in.	"	34s. 0d.	40s. 0d.

SAWN TIMBER FOR STRUTS AND SUPPORTS—

3 in. by 4 in.	from £19 per standard
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MILD STEEL RODS FOR REINFORCEMENT—

	s.	d.
$\frac{1}{8}$ in. to 2½ in. Rounds	per cwt.	8 6
$\frac{3}{16}$ in. to ½ in. Rounds	"	8 9
$\frac{1}{2}$ in. Rounds	"	9 0
$\frac{3}{4}$ in. Rounds	"	10 0

Breeze Slabs per yd. super: 2 in., 1/6; 2½ in., 1/8; 3 in., 2/-; 4 in., 2/4.

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Do. do. in columns	" "	1	5
Do. do. in beams	" "	1	7
Do. do. in floor slabs 4 in. thick	per yard super	3	8
Do. do. in floor slabs 5 in. thick	" "	4	7
Do. do. in floor slabs 6 in. thick	" "	5	6
Do. do. in floor slabs 7 in. thick	" "	6	5
Do. do. in walls 6 in. thick	" "	5	6

(Add for hoisting 3s. 6d. per yard cube above ground-floor level. Add for rapid hardening Portland Cement 2s. per yard cube.)

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" $\frac{7}{16}$ in. to $\frac{1}{2}$ in.	"	15 0
" $\frac{9}{16}$ in. to 2½ in.	"	14 0

EXTRA LABOUR TO BENDS in $\frac{1}{4}$ -in. rods, ½d.; $\frac{3}{8}$ -in. rods, 1d.; $\frac{1}{2}$ -in. rods, 1½d.; $\frac{5}{8}$ -in. rods, 1½d.; $\frac{3}{4}$ -in. rods, 1½d.; $\frac{7}{8}$ -in. rods, 2d.; 1-in. rods, 2½d.; 1½-in. rods, 3d.; 1½-in. rods, 3½d.; 1½-in. rods, 4½d.; 1½-in. rods, 6d.

EXTRA LABOUR TO HOOK BENDS: $\frac{1}{4}$ in., 1d.; $\frac{3}{8}$ in., 2d.; $\frac{1}{2}$ in., 2½d.; $\frac{5}{8}$ in., 3d.; $\frac{3}{4}$ in., 3½d.; $\frac{7}{8}$ in., 4d.; 1 in., 4½d.; 1½ in., 6d.; 1½ in., 7d.

SHUTTERING—

	s.	d.
Shuttering and Supports for Concrete Walls (both sides measured) per square	50	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft. high	per square	46 0
Do. do. in small quantities	per ft. super	0 8
Shuttering and Supports to Stanchions, average 18 in. by 18 in.	per ft. super	0 7½
Do. do. as last, in narrow widths	" "	0 9½
Do. do. to sides and soffits of beams, average 9 in. by 12 in.	" "	0 9½
Do. do. as last, in narrow widths	" "	0 10
Raking, cutting, and waste to shuttering	per ft. run	0 3
Labour, splay on ditto	" "	0 2
Small angle fillets fixed to internal angles of shuttering to form chamfer	" "	0 3

WAGES.—The rates of wages on which the above prices are based are:—Carpenters and joiners, 1/7 per hour; Carpenters working on old shuttering, 1/8; Labourers on building works, 1/2½; Men on mixers and hoists, 1/3½; Bar-benders, 1/3½.

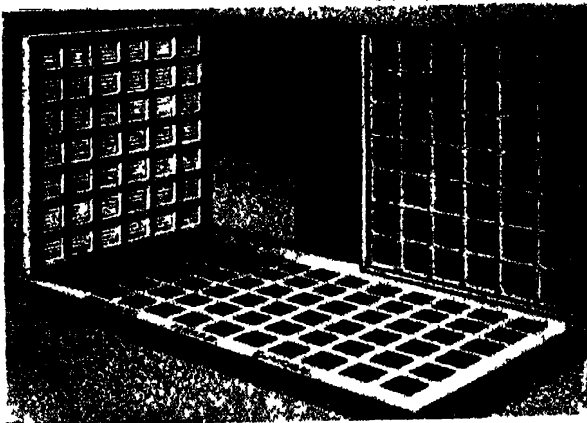
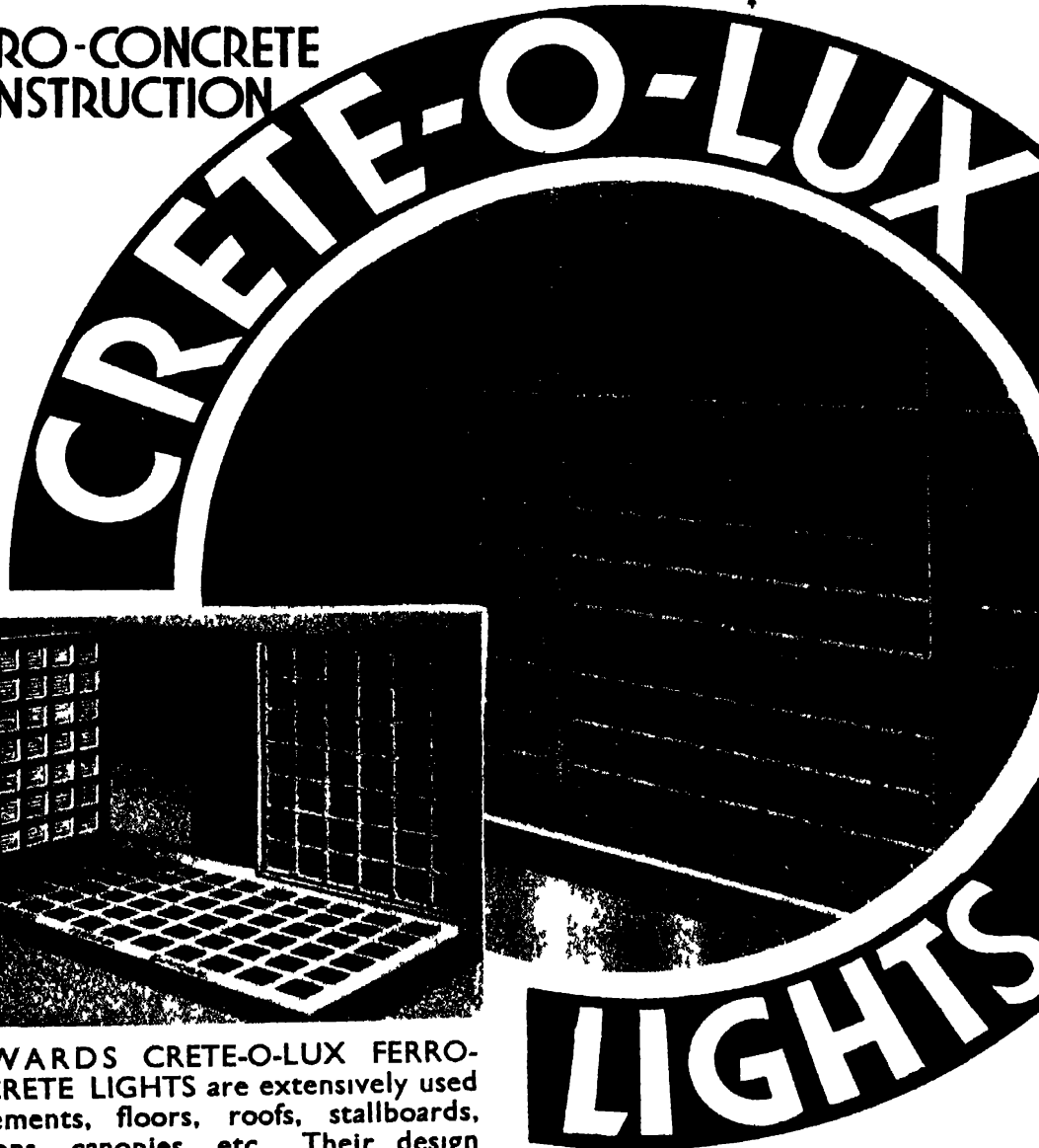
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September, 1933.



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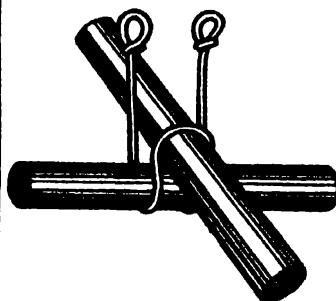
Haywards Crete-o-lux Lights perfectly fulfil the requirements of modern ferro-concrete construction while Haywards Iron-frame Lights continue in wide demand.

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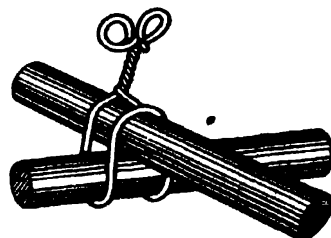
36,000 "W" glass lenses covering 4000 sq. ft. are used in the Crete-o-lux lights supplied by Haywards to Shell-Mex House.

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Prospective New Concrete Work.

ALTRINCHAM.—*Bridge.*—Subject to a grant, the Cheshire C.C. proposes to reconstruct the Altrincham canal bridge, at an estimated cost of £24,000.

ARMAGH.—*Sewerage.*—The U.D.C. is to apply for a loan of £30,000 for a sewerage scheme.

BIDEFORD.—*Water Supply.*—The T.C. has applied for sanction to borrow £3,070 for water supply works.

BISHOP AUCKLAND.—*Bathing Pool.*—The U.D.C. proposes to construct a bathing pool near Dam Head.

BLACKPOOL.—*Swimming-Bath.*—The C.B. proposes to construct a swimming-bath for children at the junction of Park Road and Glastonbury Avenue.

BOOTLE.—*Flood Prevention.*—The Bootle T.C. is considering a proposal for the construction of relief sewers.

BOSTON.—*Water Supply.*—The T.C. proposes to construct water-supply works, which includes the construction of pumping stations at Ulceby and Revesby, at an estimated cost of £86,500.

BRIDGWATER.—*Water Supply.*—The R.D.C. has applied for a loan of £35,618 for water-supply works.

CAISTON.—*Water Supply.*—The R.D.C. is considering a water-supply scheme for the Market Rasen district, estimated to cost £10,000.

CARNLOUGH.—*Sewerage Works.*—The Larne R.D.C. has applied for a loan of £3,000 for sewerage works at Carnlough.

CASTLEFORD.—*Market.*—The U.D.C. proposes to erect a new fish and meat market, estimated to cost £7,500.

CHINGFORD.—*Swimming Pool.*—The C.C. is considering the construction of a swimming pool and lido.

CHIPPING NORTON.—*Swimming Pool.*—The T.C. is considering a proposal to construct a swimming pool.

CLAYTON WEST.—*Bridge.*—The U.D.C. proposes to reconstruct Langley bridge.

COALPOOL.—*Sewerage.*—The Wallsall T.C. is to proceed with the Coalpool and Harden sewerage scheme, estimated at £7,721, subject to a loan being sanctioned.

COLESHILL.—*By-pass Road.*—The Warwick C.C. has recommended the construction of a by-pass road at Coleshill, subject to the M.T. making a grant of 60 per cent towards the estimated cost of £50,000.

DURHAM.—*Roads.*—The Durham C.C. report that the M.T. have made grants of 85 per cent towards the cost of widening the Coatham Munderville-Darlington section of the Great North Road, estimated at £46,424, and of works on the Rushyford Road section estimated at £68,225.

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CONCRETE

EASTBOURN - *Bathing Pool etc* - The Corporation is considering a scheme for the development of Redoubt, including the construction of a bathing pool estimated to cost £65,000.

EAST BARNET - *Sewage Disposal* - It is announced that arrangements are now being completed for the construction of sewage disposal works at East Barnet sewage farm at Brunswick Park. The total cost of the complete scheme is estimated at £135,000.

ENNISKILLEN - *Swimming Bath* - The U.D.C. is considering a proposal to construct a swimming bath.

HERTFORDSHIRE - *Road* - The C.C. proposes to widen the main road between Hockerill and Stanstead at Bishop's Stortford, at an estimated cost of £11,816.

HODNET - *Water Supply* - The Market Drayton R.D.C. is to apply for sanction to borrow £3,600 for a water supply for Hodnet village.

ILKLEY - *Bathing Pool* - The U.D.C. is to apply for a loan of £5,000 to construct a bathing pool.

LAMBOURN - *Water Supply* - The Hungerford R.D.C. is to prepare a scheme



for a water supply for Lambourn and district at an estimated cost of £21,304.

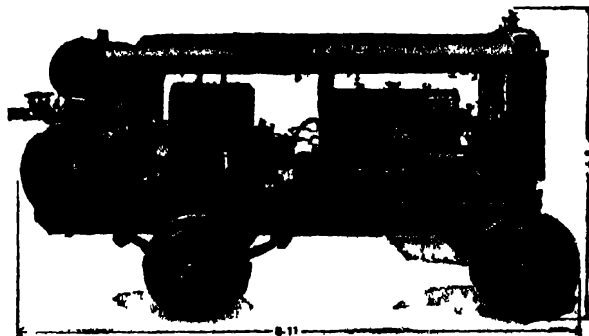
LANCASHIRE - *By pass Road* - Sanction has now been given by the M.T. for the completion of the by pass road between Preston and Blackpool.

LANGFORD - *Sewage Disposal* - It is proposed to construct joint sewage disposal works for the parishes of Langford, Shefford, Henlow and Clifton at an estimated cost of £60,000.

LEICESTER - *Abattoir* - The L.C. is considering the construction of an abattoir.

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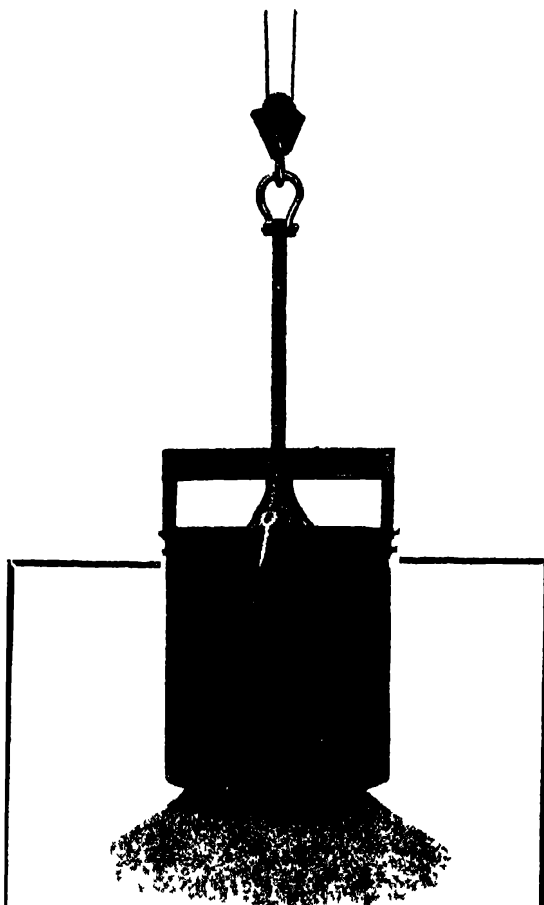
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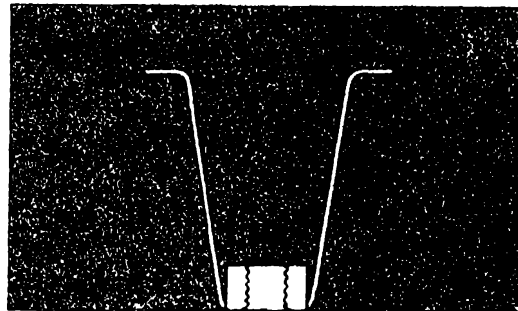
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From all quarters of the globe reports are received of work of an unusual on which "Ferrocrete" has been used. Here is one from China. In order to make a deep mid-stream channel at K'ungling'an on the Upper Yangtse, it was found necessary to remove considerable rock formations which were only partly exposed at dead low water level. It was decided last winter, therefore, to build reinforced concrete platforms on the rocks in the form of "ships," rising to a height of 11 ft. above local zero level. For this purpose, "Ferrocrete" rapid-hardening Portland Cement was specified because a saving of time was so important in view of a possible early rise in the water level. Actually, shot firing was carried out inside the "ships" within 4 days of pouring the concrete. Although subject to such violent treatment, the concrete showed no sign of cracking and the shafts were successfully sunk. One shaft had to be lined with "Ferrocrete" to lessen the flow of water through the fissures in the rock. The charges were finally laid, the rocks demolished and a new low water channel for steamers created. Concrete made with "Ferrocrete" is as hard and as strong in four days as concrete made with ordinary Portland Cement is in four weeks.

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LINCOLN Water Supply The R D C is to apply for a loan of £ 10,000 for a water supply scheme

LLANDUDNO — Sea Defence The L D C is considering a scheme for sea defence works on the North Shore

ILLECHRYD Bridge The Cardigan shire C C has decided to carry out reconstruction work in concrete on the Illechryd bridge over the river Teify at an estimated cost of £ 5,000

LONDON (Woolwich) Road It is reported that work will commence shortly on the Eltham Lee section of the circular road from Woolwich to Batterssea and that the L C C has decided to spend £ 10,000 on the scheme

TOWER WOLVERCOTE Bridges The Oxfordshire C C is considering the proposed construction of two canal bridges and railway bridge at Tower Wolvercote

LYNN Church Road The L C has been informed that it is proposed to construct a concrete road from Greatstone to the Pilot at an estimated cost of £ 10,000

MARKET RASEN Water Supply — The Lincoln R D C is to apply for sanction to proceed with the Market Rasen water supply scheme estimated to cost £ 10,000

“ ESTIMATING & COST KEEPING FOR CONCRETE STRUCTURES ”

By A E WYNN, B Sc., A M Am Soc C E

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Estimating and Cost Keeping**

Mr Wynn has had an extensive experience of reinforced concrete construction both in England and the United States and after close study of practically every method of estimating and cost keeping now in use he has devised and adopted for his own business a system that is particularly suitable for concrete and reinforced concrete constructors and which is also largely applicable to general contracting and building businesses

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MATLOCK — *Bathing Pool* — The U D C proposes to construct a bathing pool

MORFICAMBI — *Sewerage* — The T C has decided to adopt a new sewerage scheme which together with improvements to the existing works is estimated to cost £200 000

NEWTOWNBRI DA — *Sewerage* — The Hillsborough R D C is to apply for a loan of £6 030 for the Newtownbreda sewerage scheme

NORTON — *Pumping Stations etc* — The U D C is considering a proposal to construct three pumping stations at an estimated cost of £29 150 and a joint sewerage scheme for Malton and Norton

PENZANCE — *Bathing Pool* — The I C is to proceed with the construction of a bathing pool at an estimated cost of £14 000

PETERBOROUGH — *Swimming Bath etc* — The I C is to submit to the M H a scheme for the construction of a swimming bath and the erection of a bus station estimated to cost £14 990

PORTOBELLO — *Pier etc* — It is reported that a company is being formed to erect a pier and esplanade at Portobello Edinburgh

PULBOROUGH — *Bridge* — The Worthing M C is considering a proposal to construct a reinforced concrete bridge over the river Arun at Pulborough at an estimated cost of £13 000



SOLWAY FIRTH — *Canal* — The Carlisle C B C proposes to construct a canal joining Solway Firth with the Tyne

STLVINAGE — *Swimming Bath* — The U D C has applied for sanction to borrow £2 500 for the construction of an open air swimming bath

STONE — *Swimming Bath* — The U D C proposes to construct a swimming bath at an estimated cost of £1 700

STONEHAVEN — *Swimming Pool* — The I C is to proceed with the construction of a swimming pool at an estimated cost of £5 500

TRALEE — *Concrete Viaduct* — The Harbour Board proposes to construct a concrete viaduct at Lemit port subject to a grant being obtained towards the estimated cost of £48 000

TRITLINGTON — *Road Works* — The Northumberland C C has decided subject to a grant to proceed with work on the Great North Road between Irlington and Irlton at an estimated cost of £119 933

WARRIOR WITH STALORTH — *Swimming Bath* — The U D C is proposing the construction of a swimming bath

WORCESTER — *Flood Prevention* — The River Severn Catchment Board proposes to construct flood prevention works in the lower reaches of the river Severn

WORKINGTON — *Swimming Baths* — The T C proposes to construct swimming baths

YIFSWLEY — *Bathing Pool* — The West Drayton U D C has applied for sanction to borrow £2 800 for the construction of an open-air bathing pool

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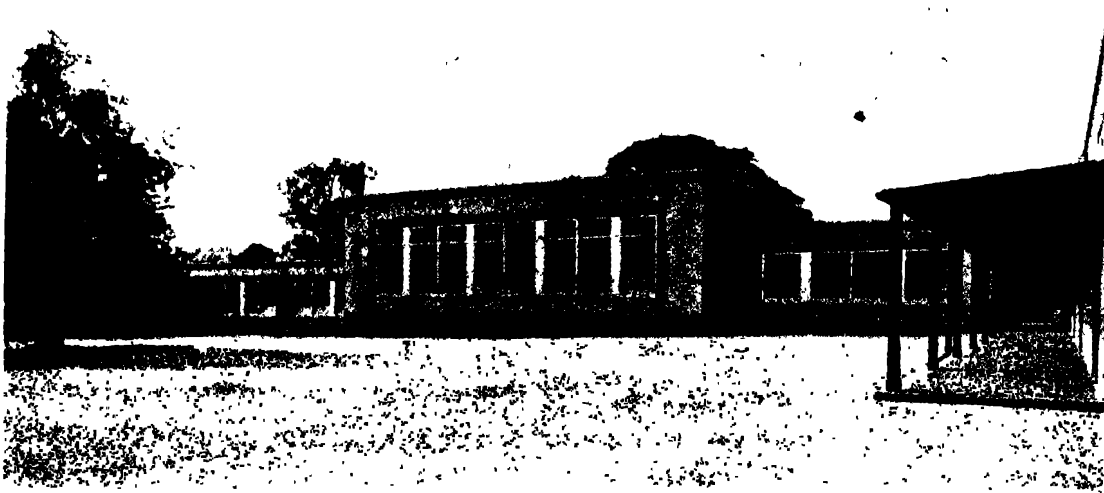
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Work was started in August, 1932, and the school opened in May, 1933.

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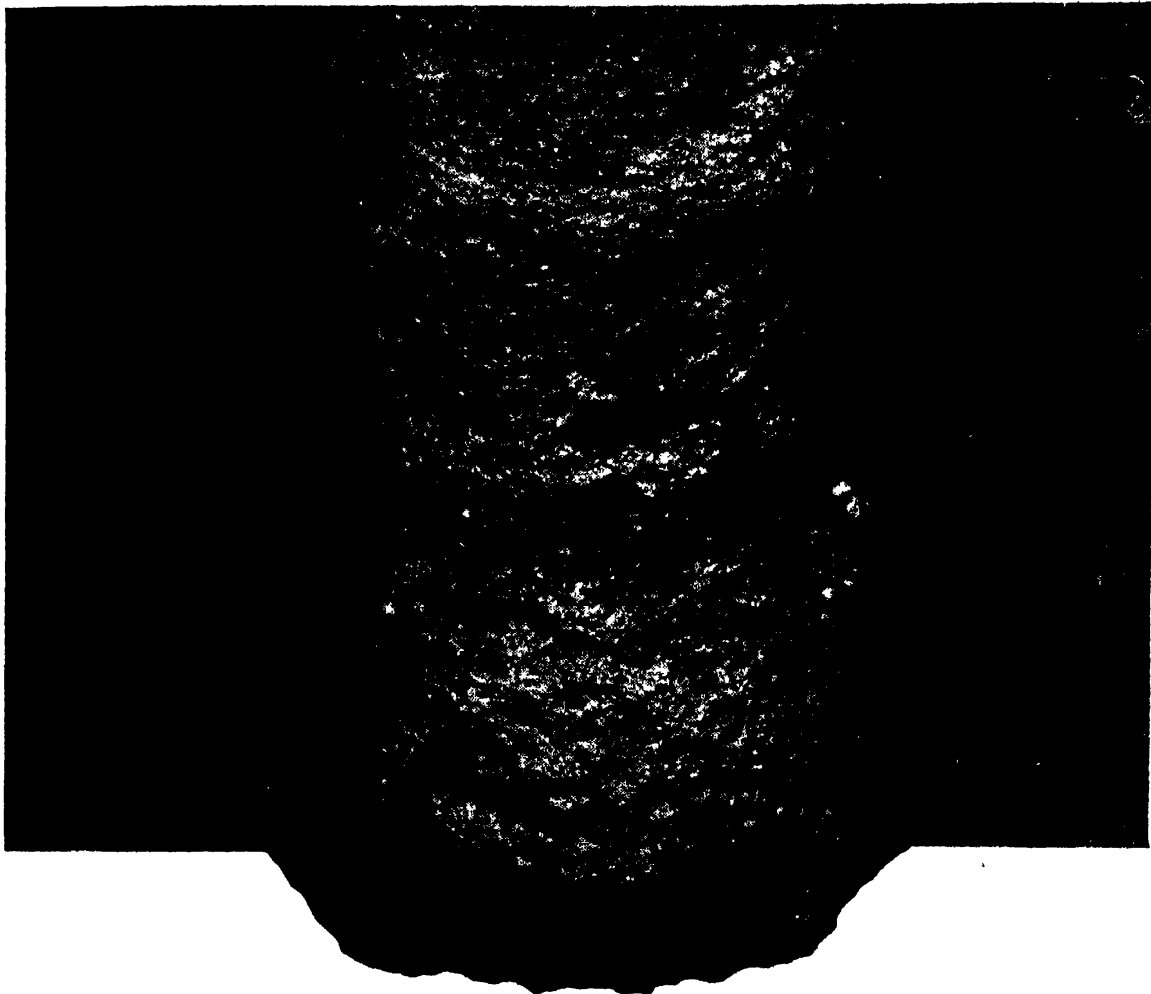
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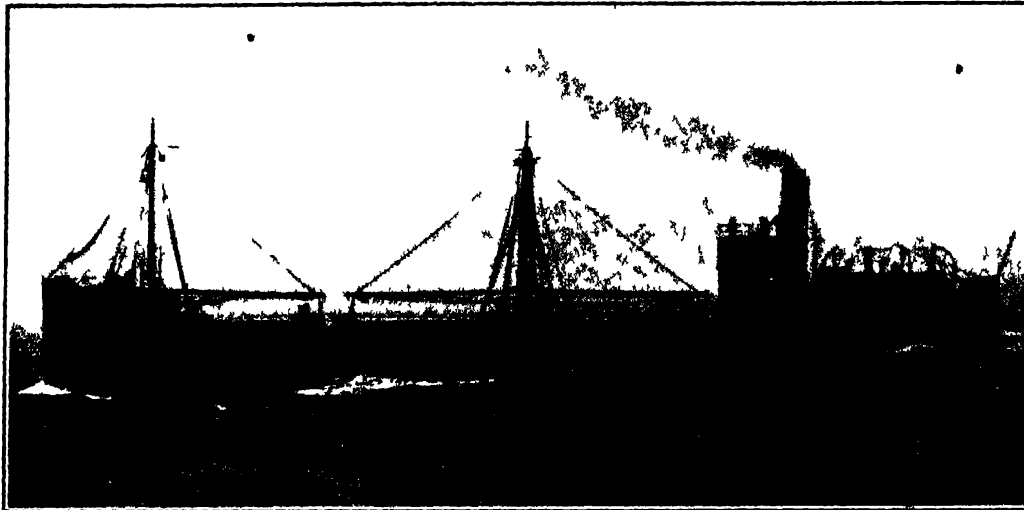
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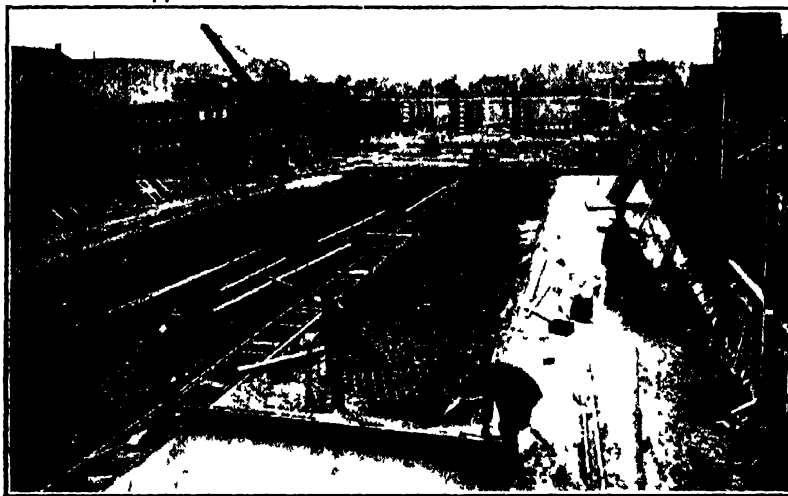
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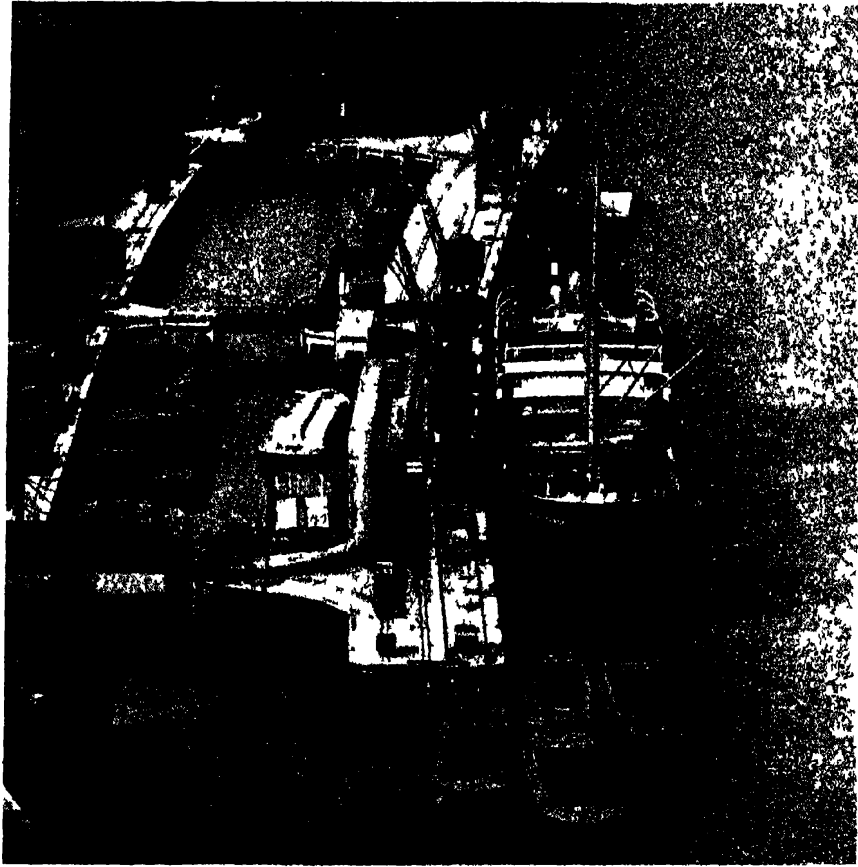
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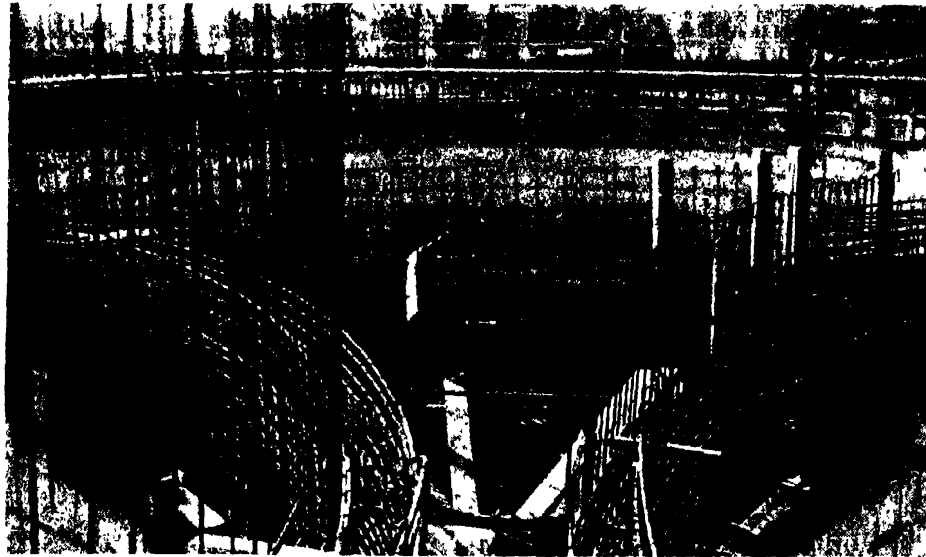
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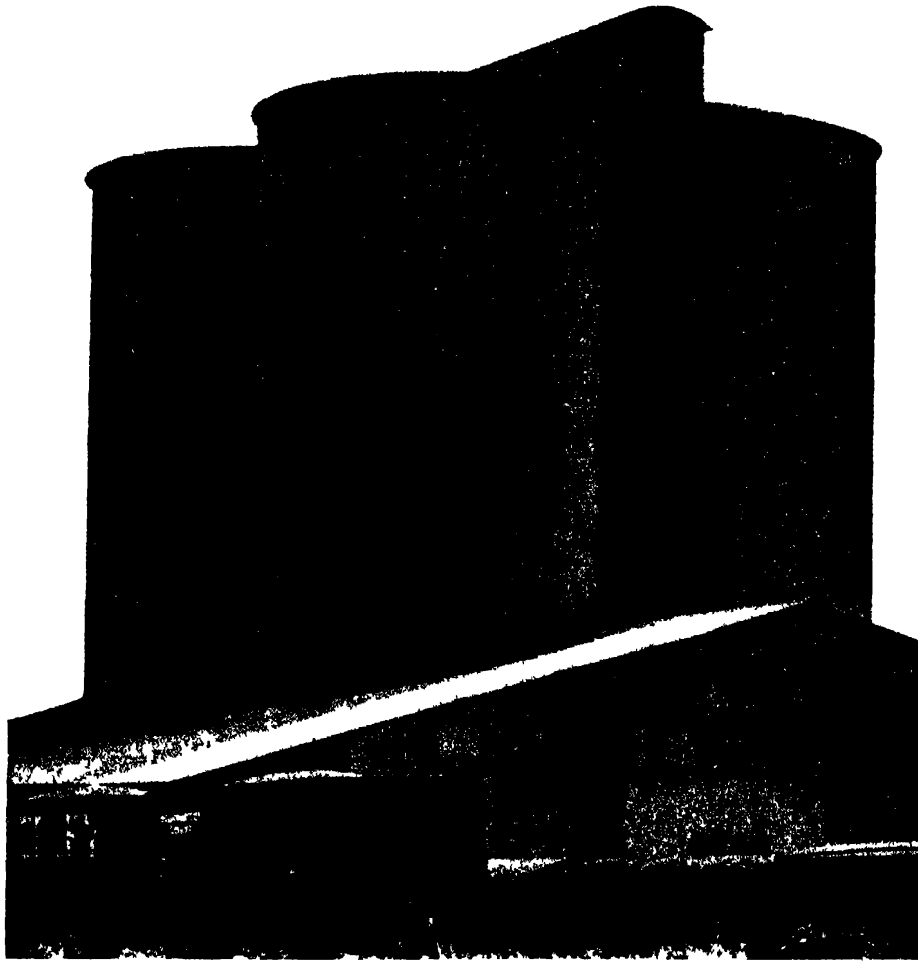
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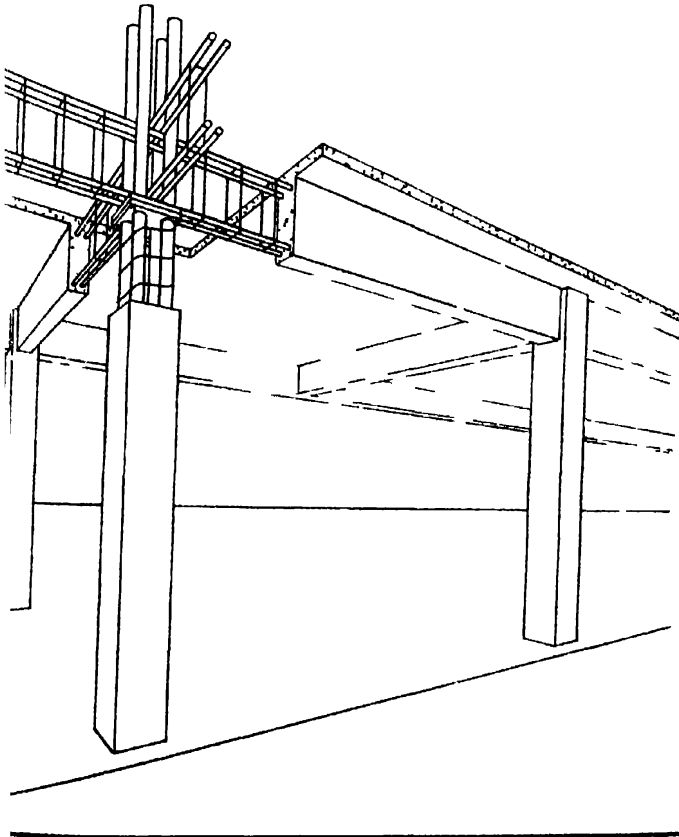
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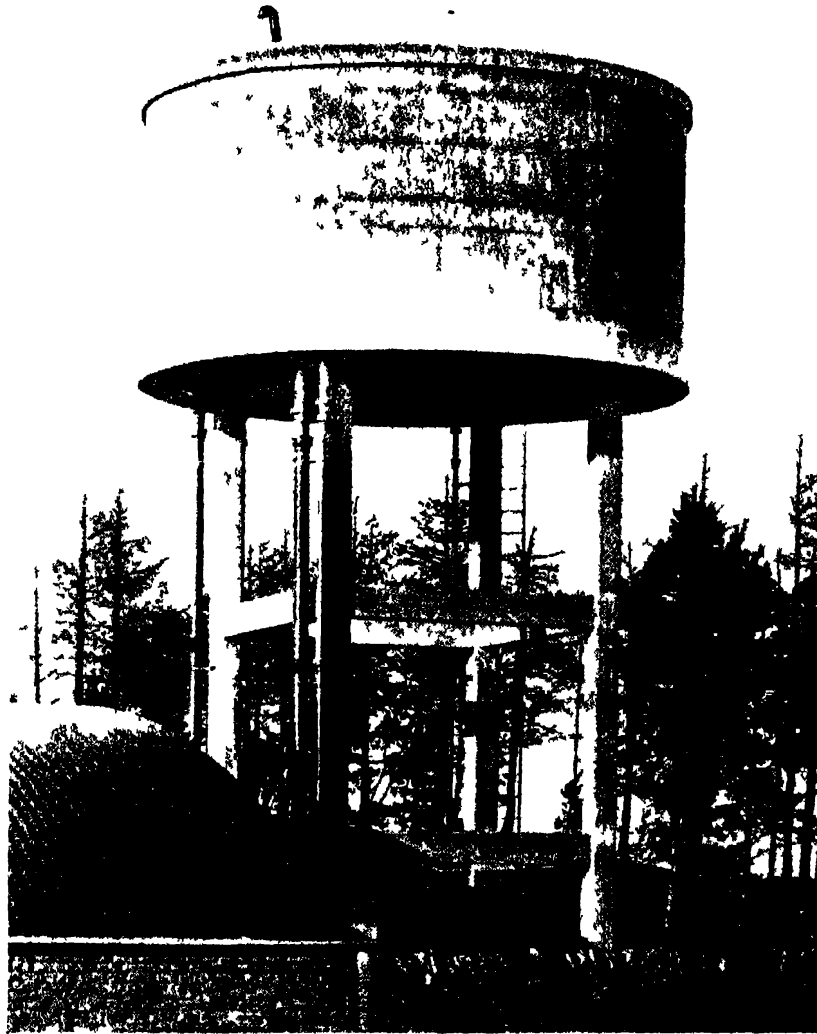
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Water Tower at Chipping Norton (Oxon)

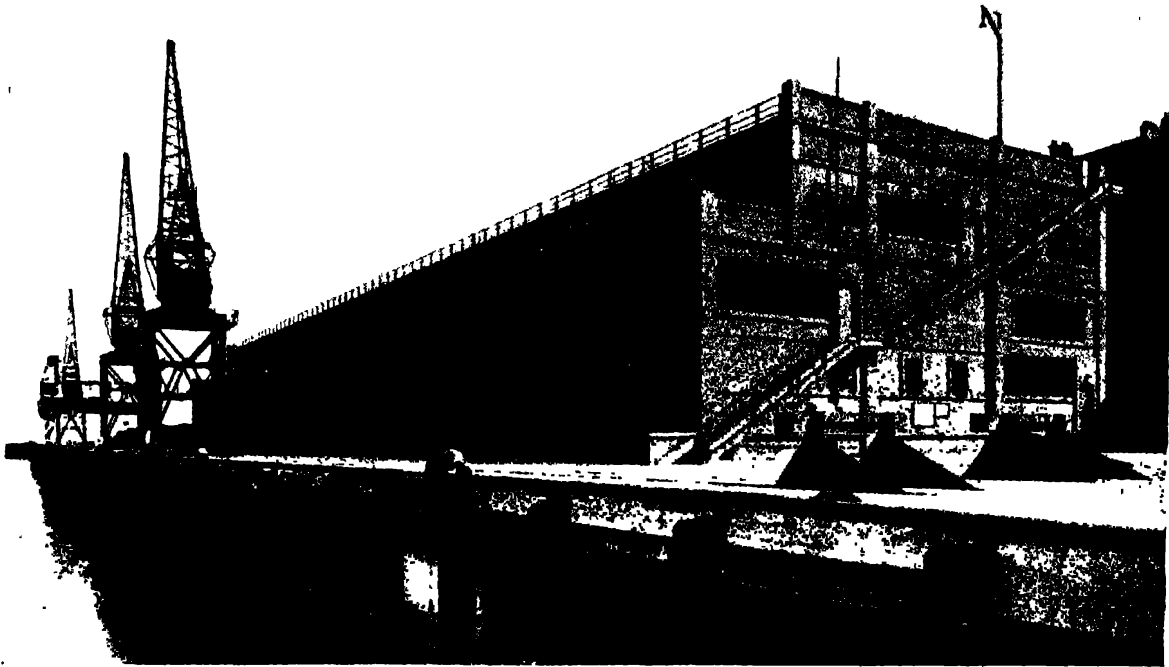
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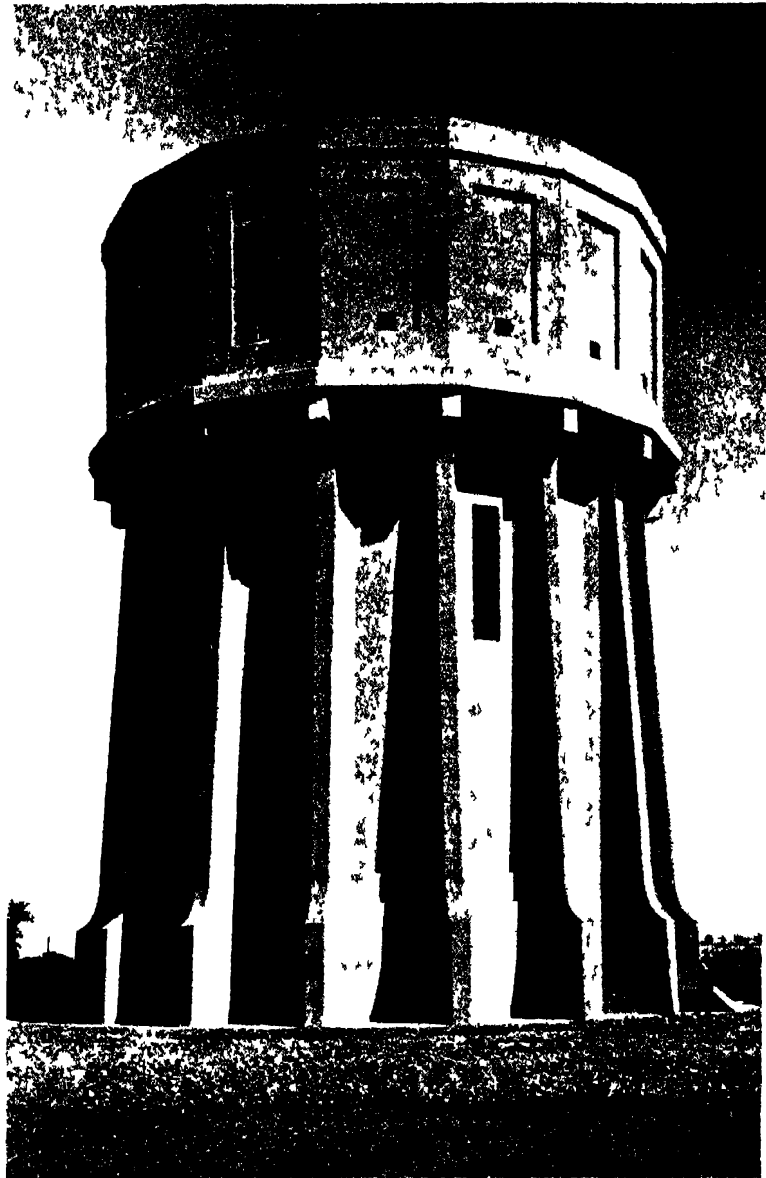
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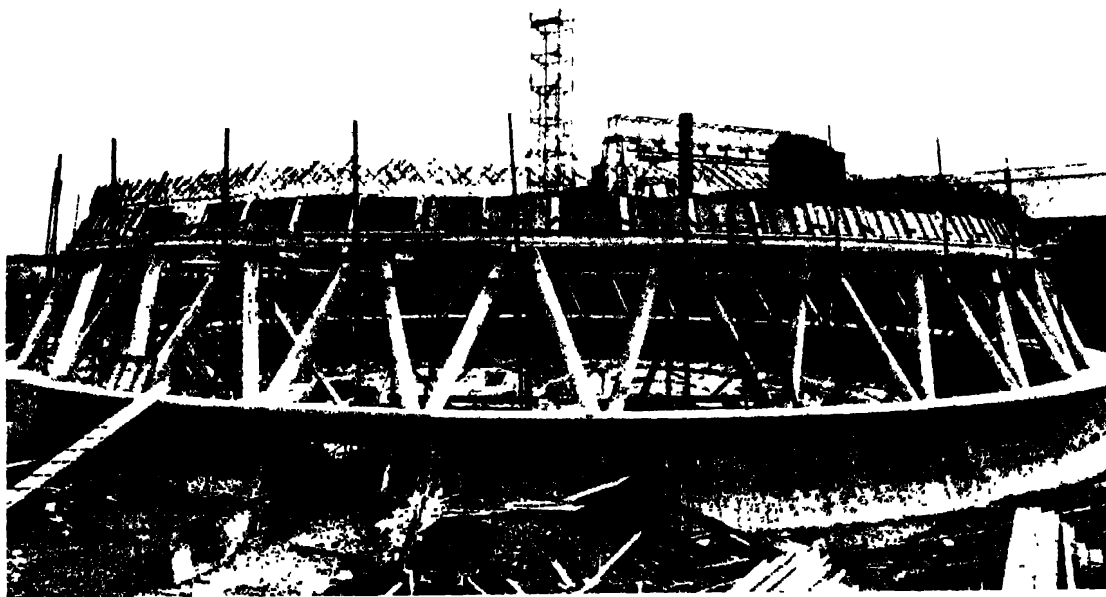
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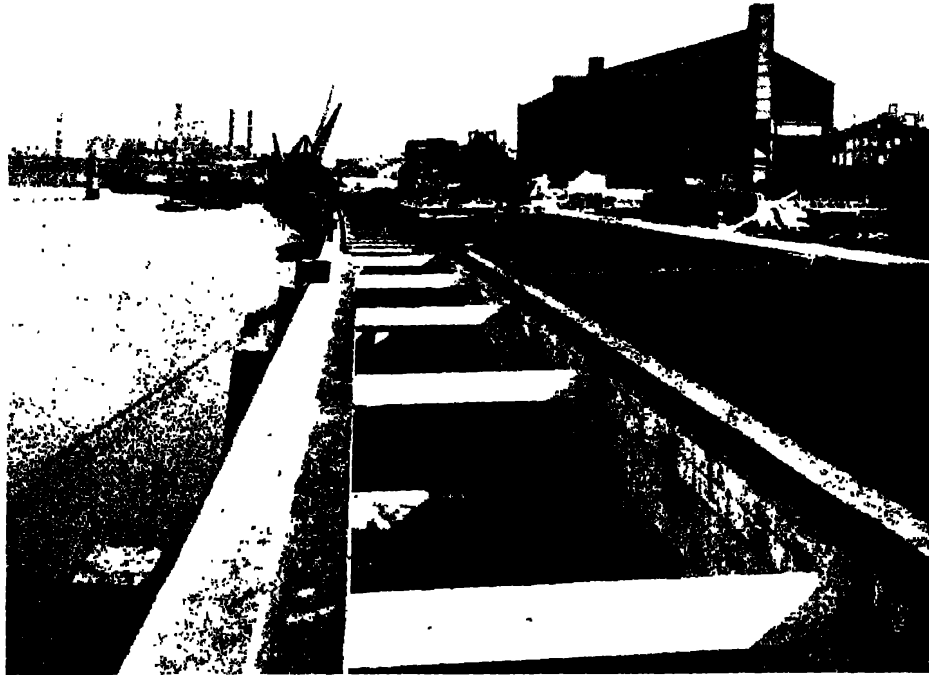
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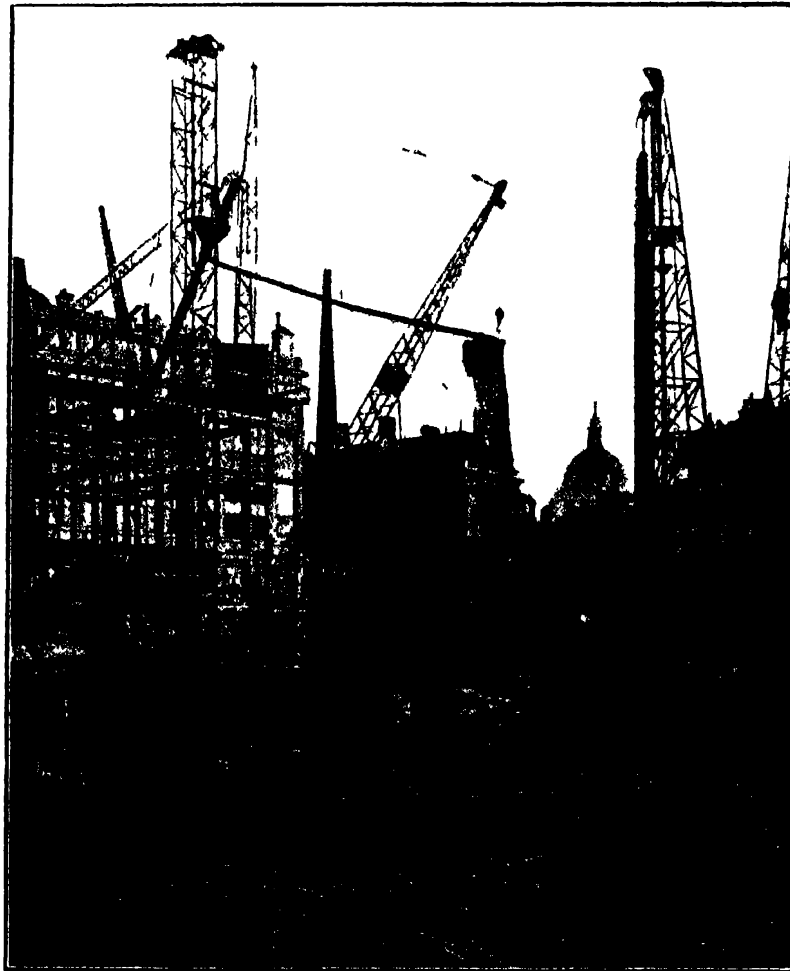
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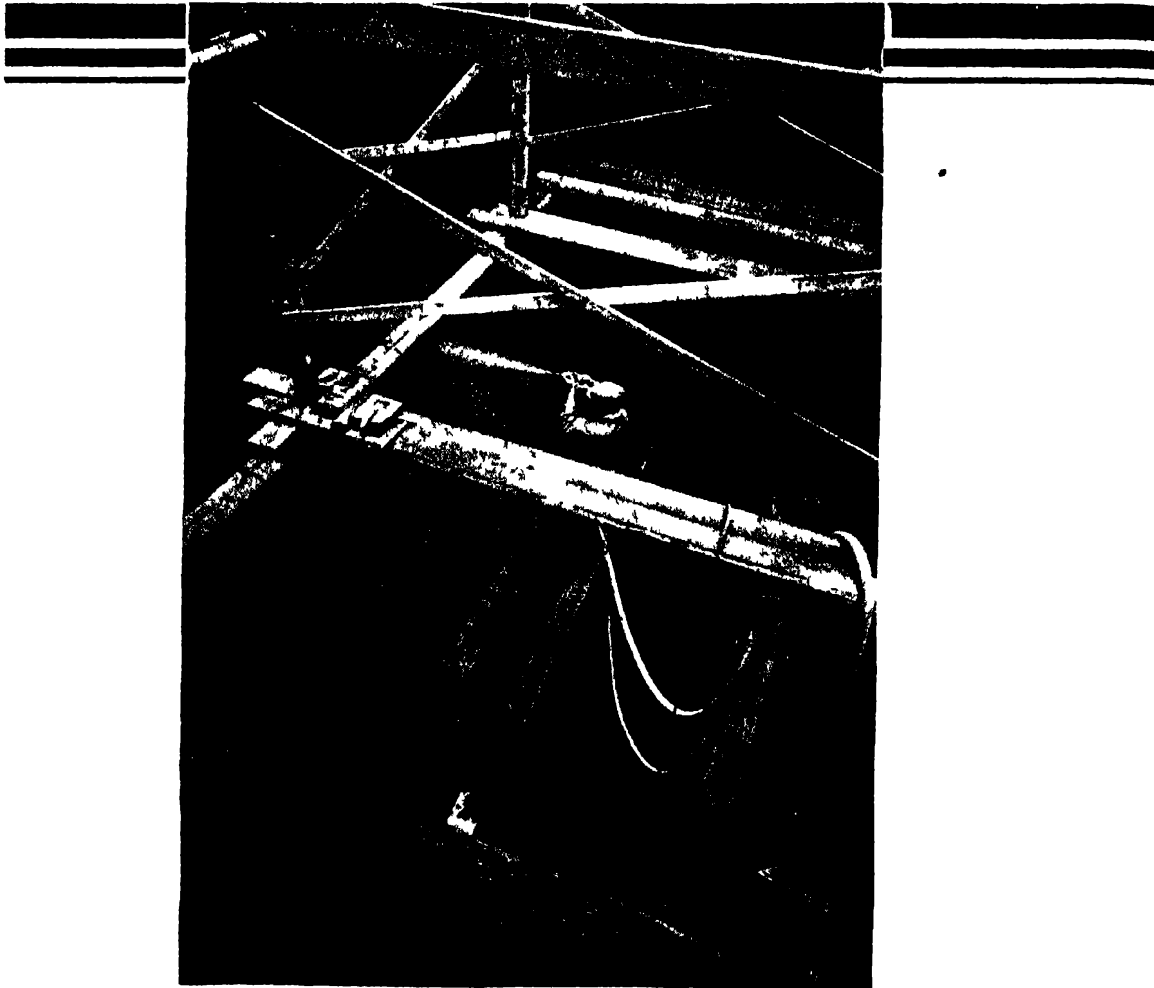
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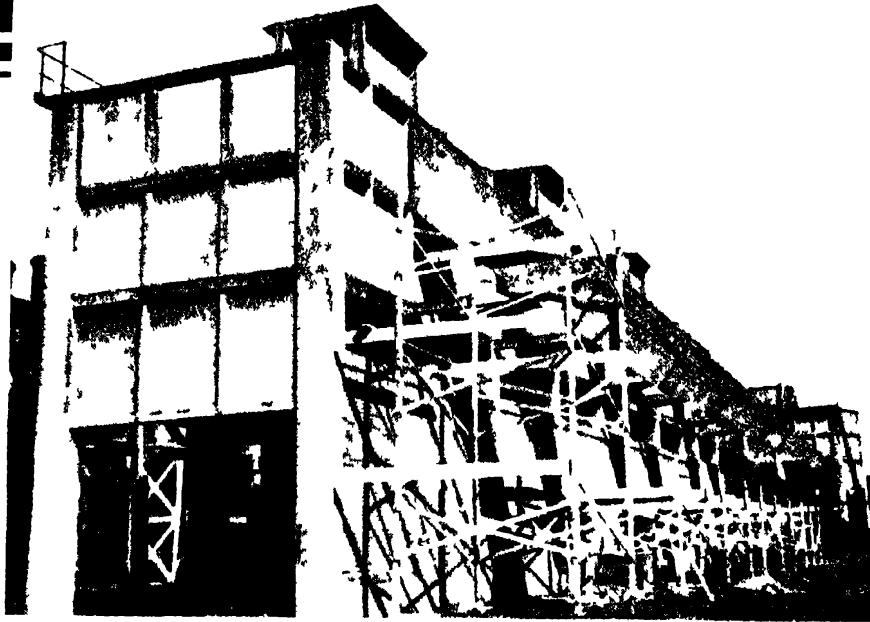
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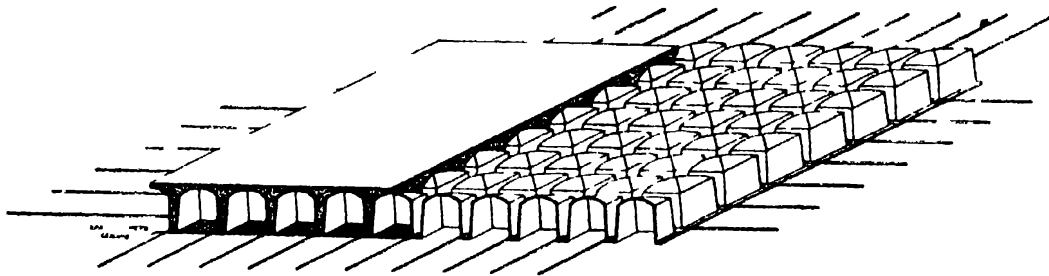
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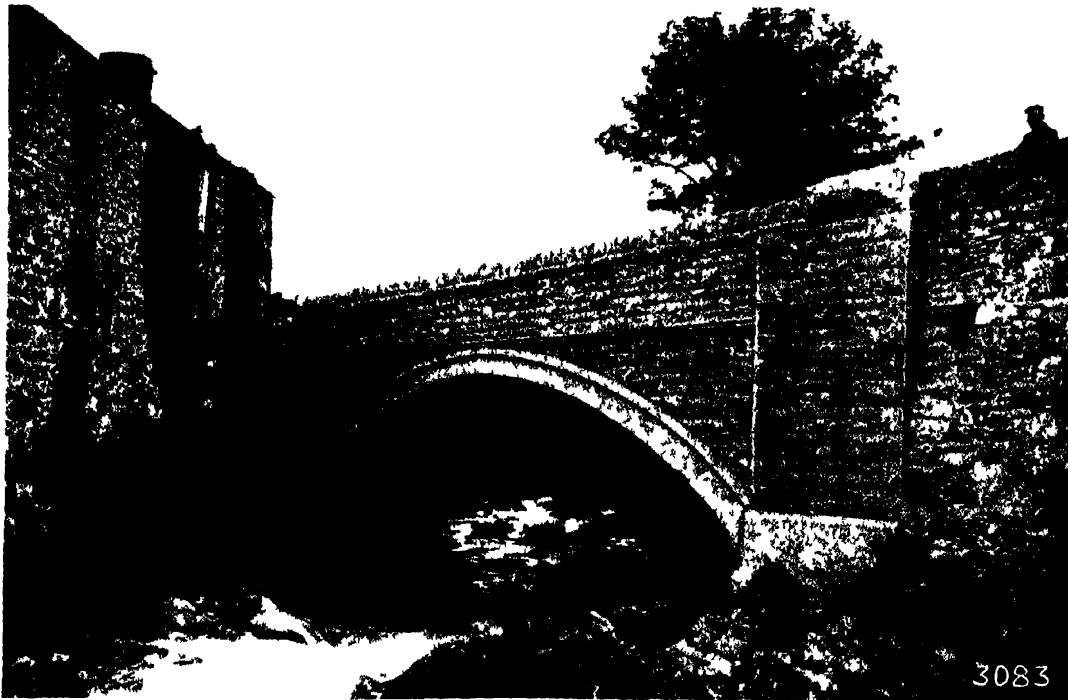


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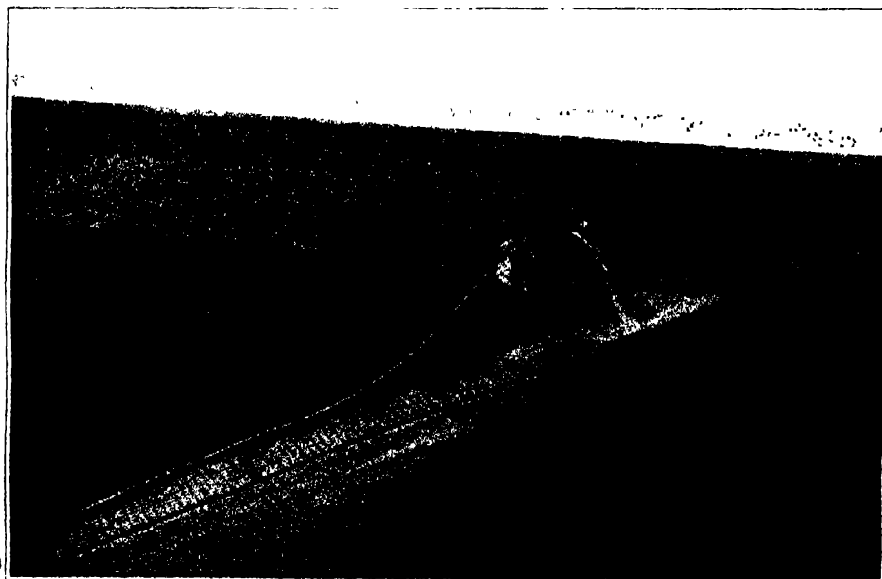
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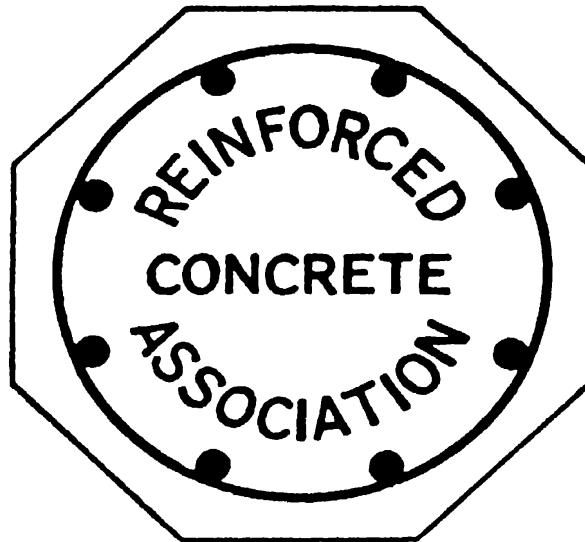
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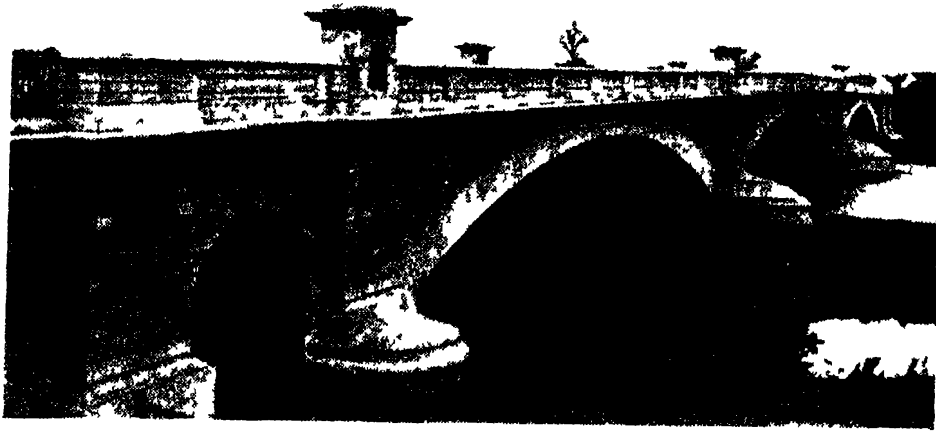
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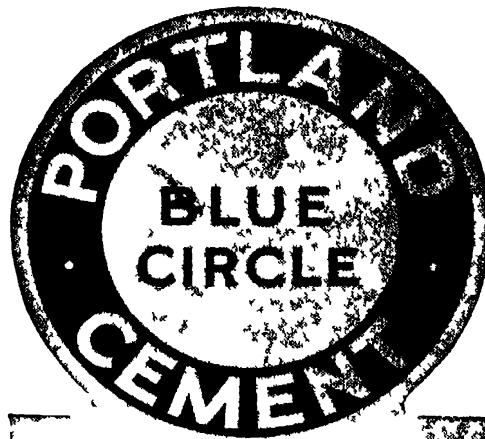
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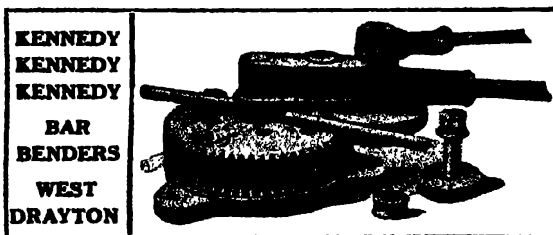
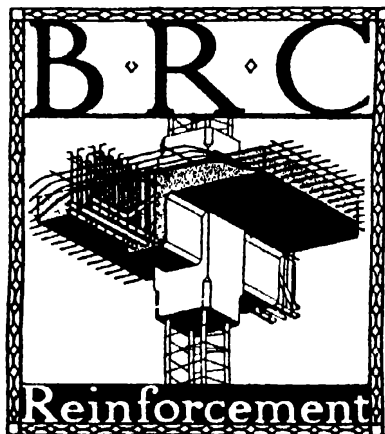
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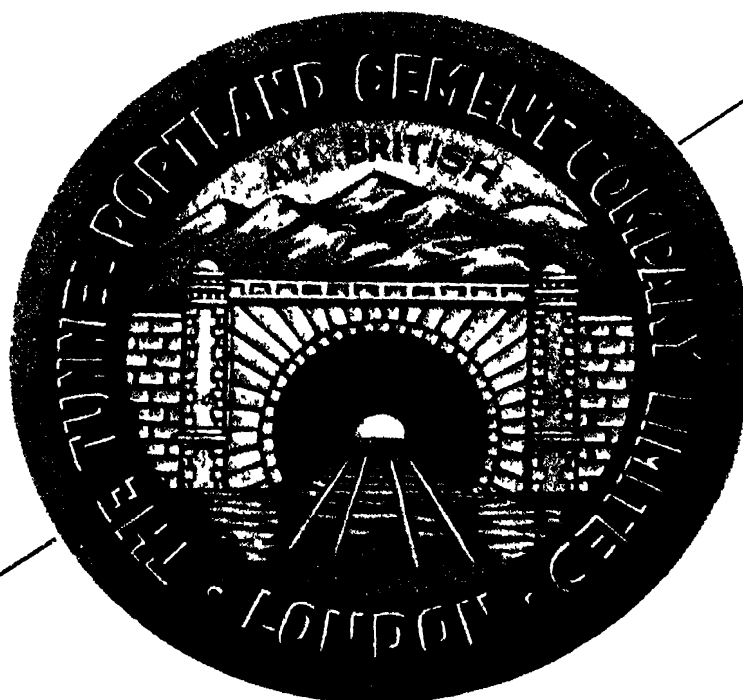
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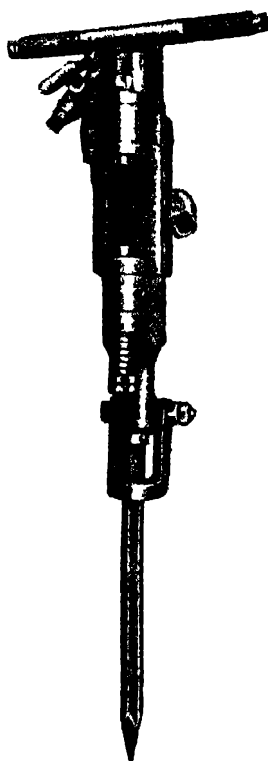


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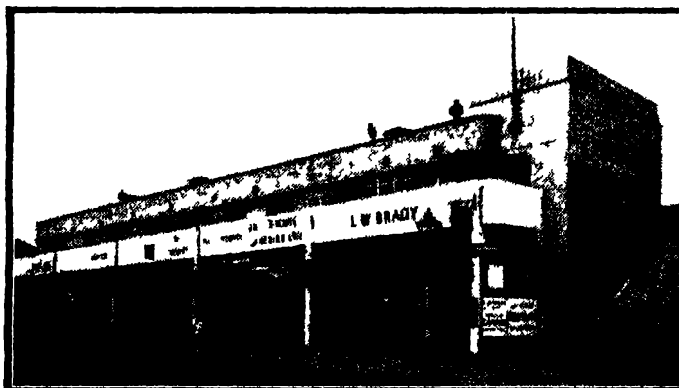
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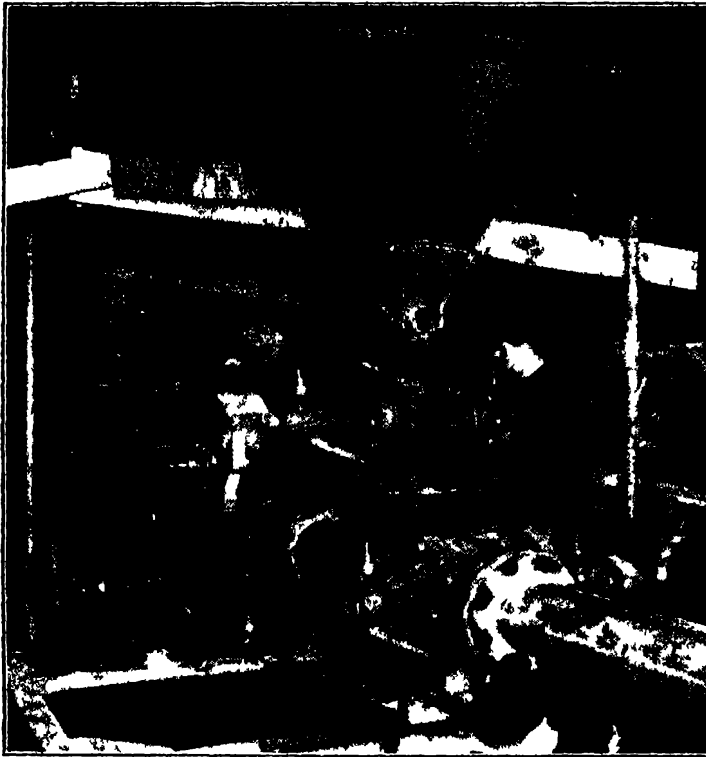
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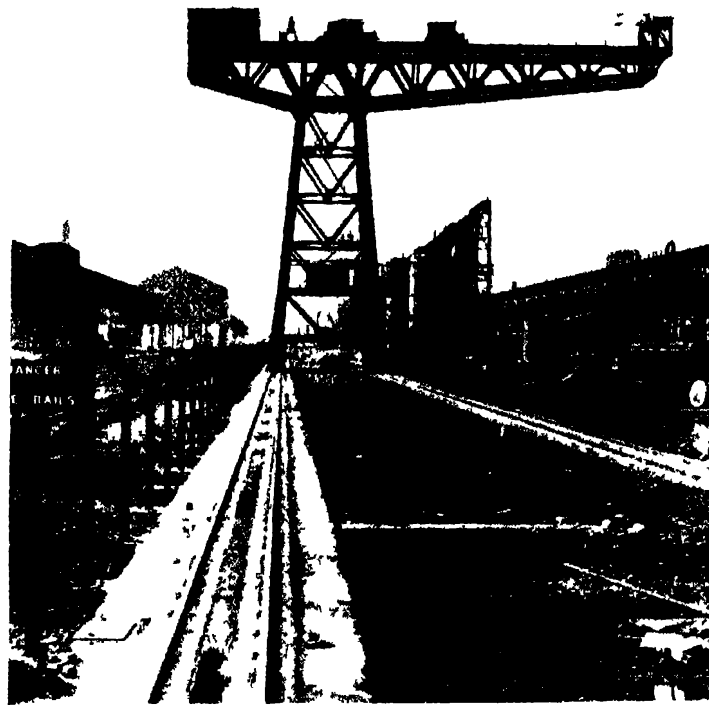
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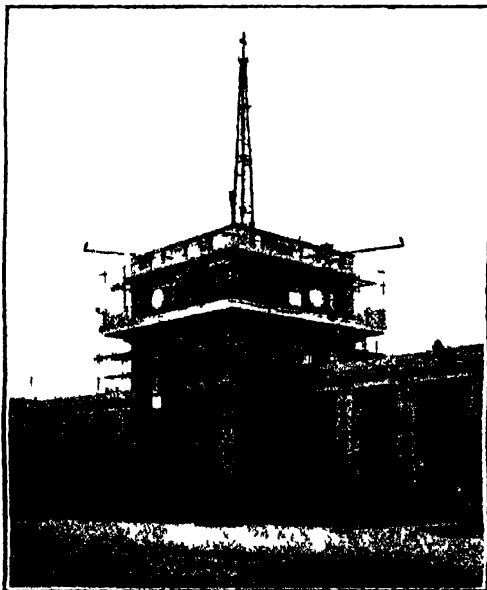
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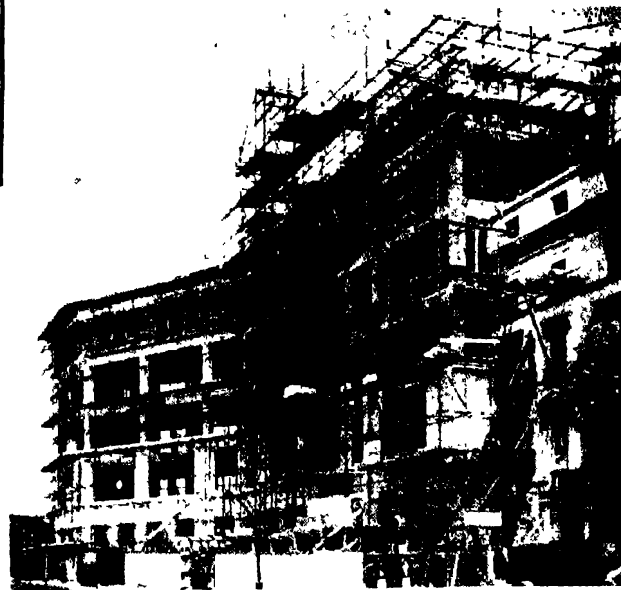
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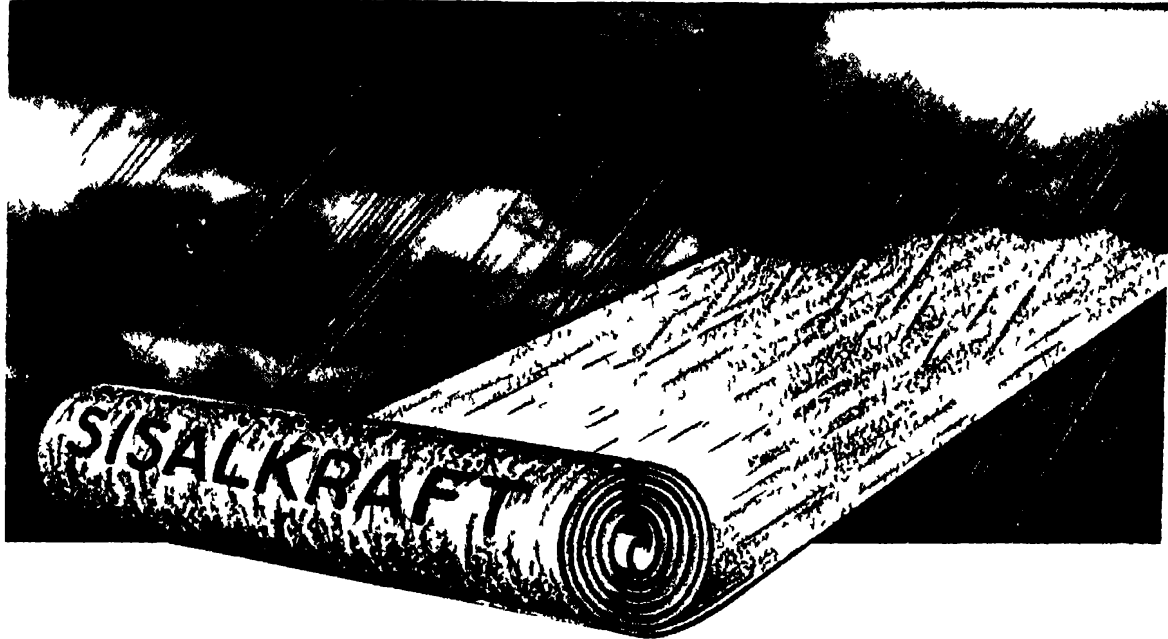
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CONCRETE AND CONSTRUCTIONAL ENGINEERING

Volume XXX No. 1

JANUARY 1935

EDITORIAL NOTES.

The London County Council Regulations.

SOME CONCESSIONS GRANTED DURING 1934

DURING the past year reinforced concrete engineers in the London area have welcomed a number of important concessions granted by the London County Council which enable designers to take advantage of the improvements in the quality of Portland cement, the fuller knowledge of methods of making high strength concrete, and the more accurate estimation of the loads superimposed on structures of varying types. Although it has not yet been found possible to issue new regulations for reinforced concrete, the Council's powers of waiver can be exercised to avoid the stringent requirements of the existing rules and place reinforced concrete design and construction in London on a basis more nearly in accord with that existing outside Great Britain. The waivers which have been granted are a step towards more scientific and therefore more economical design, and their general adoption throughout Great Britain is only a matter of time.

Two important factors in the design of an economical framework to support a given load are the correct estimation of the superimposed load and the maintenance of the weight of the structure or the dead load at the lowest possible level. Insofar as the former is concerned it is clearly in the public interest that the estimate should err on the side of safety, but at the same time the owner of the structure must not be penalised by being compelled to use an unnecessarily high quantity of building material to carry the load. In the past heavy expenditure has often been required to meet the regulations of public authorities. For instance, the Board of Trade rule specifying a horizontal wind pressure of 56 lb per square foot on railway bridges, which was adopted after the Tay Bridge disaster in 1879, was undoubtedly very severe in all but the most exposed situations and its adoption has greatly increased the cost of many works. At a later date it was found that the relation between the velocities of the wind and the anemometer cups had been overestimated, but the wasteful effect of the rule continued. In building construction also the tendency in the past was to specify heavy loadings, and it was not until the adoption of the Code of Practice for the use of structural steel that this handicap in design was removed by the London County Council. For reinforced concrete construction new regulations have not yet been issued by this body, but it is now possible to obtain considerable concessions by waiver. In this way the floor loads contained in the Code of Prac-

tice for structural steel may be permitted in reinforced concrete work. Moreover, in rooms to be used for domestic purposes, a loading of 40 lb. per square foot (exclusive of any allowance for partitions) is permitted instead of the 70 lb. required by the existing regulations. All floors must be capable of carrying in any position on an otherwise unloaded floor a concentrated load, in the case of floors of rooms used for domestic purposes, of $\frac{1}{2}$ ton. This alternative concentrated load is to be considered in each case as equally distributed over a floor area of 2 feet 6 inches square. The reactions due to these alternative concentrated loads need not be allowed for in calculating the loads on pillars and foundations.

Instead of the 112 lb. per square foot specified in the 1909 Act a superimposed load of 80 lb. per square foot is permitted on floors used wholly or principally for the purpose of a retail shop, provided that all floor beams are capable of carrying in any position on an otherwise unloaded floor a concentrated load of 2 tons. Office-floor loads are also reduced by waiver from 100 lb. per square foot to 50 lb. per square foot plus 20 lb. per square foot for partitions, subject to (1) all floor constructions between the supporting beams of the skeleton framework being calculated to support a superimposed load of 100 lb. per square foot, and (2) the additional allowance of 20 lb. per square foot for partitions being increased if necessary.

In calculating the total loads to be carried on foundations, pillars, and walls, the reductions in the superimposed loads (other than those on the roof and top floor) have been doubled. This increased allowance is to some extent counter-balanced by the reductions made in the floor loads.

Reference has already been made to the importance of a small dead load in economical construction. In large structures the stresses due to dead load generally bear a high proportion to the total stresses on account of the great weight of the members. Increased working stresses are helpful in reducing dead weight and when combined with a suitable design have an important effect on the total cost of a structure. Their relative importance, however, depends to some degree on the cost of materials and formwork. Waivers have been granted by which a maximum compressive stress of 750 lb. per square inch is permitted in 1 : 2 : 4 concrete, subject to an ultimate compressive resistance of 3,000 lb. at four months, and maximum stresses of 1,000 lb. and 930 lb. in 1 : 1 : 2 concrete, subject to an ultimate strength of 4,000 lb. at four months.

Under the existing regulations no provision is made for flat-slab or mushroom floor construction, a type of structure which has been very widely adopted outside Great Britain for offices and warehouses. Owing partly to the lack of official recognition flat-slab construction in this country has been almost entirely confined to reservoir roofs and a certain class of piled foundation, but the grant of waivers by the London County Council for structures of this type will probably give rise to its more frequent use. In the November issue of *CONCRETE AND CONSTRUCTIONAL ENGINEERING* a flat-slab warehouse was described, which is believed to be the first of its kind erected in London. We understand that a waiver has been granted for a building with flat-slab floors and roof in which a 1 : 1½ : 2½ concrete with a compressive strength of 5,800 lb. per square inch will be used under maximum compressive flexural stresses of 1,450 lb. per square inch and the tensile stress in the steel will be 18,000 lb. per square inch.

Ground-water Lowering and Chemical Consolidation of Foundations.

By H. J. B. HARDING, B.Sc., A.M Inst.C.E.

A LARGE rebuilding scheme is in progress for Messrs Bentalls at Kingston which will provide a steel frame building faced with brick and stone extending for 626 ft along Wood Street and 230 ft facing Clarence Street. Sir Aston Webb & Son are the architects for the whole scheme of reconstruction. The first two portions in Wood Street were completed by Messrs John Mowlem & Co. Ltd. in 1931 and 1932. The building is founded on waterlogged ballast and a concrete raft tanked with asphalt in which the new stanchion grillages are bedded carries the building. Considerable difficulty was encountered in building the first two sections owing to the quantity of water in the fine sandy ballast as the river Thames is only 100 yd. to the west of the site.

The extension described in this article is on an important corner site at Wood Street and Clarence Street at present occupied by Messrs Bentalls' original premises (*Fig. 2*) built in the last century consisting of three buildings of poor quality brickwork strengthened in parts with steel beams and with a number of steel and cast iron columns.

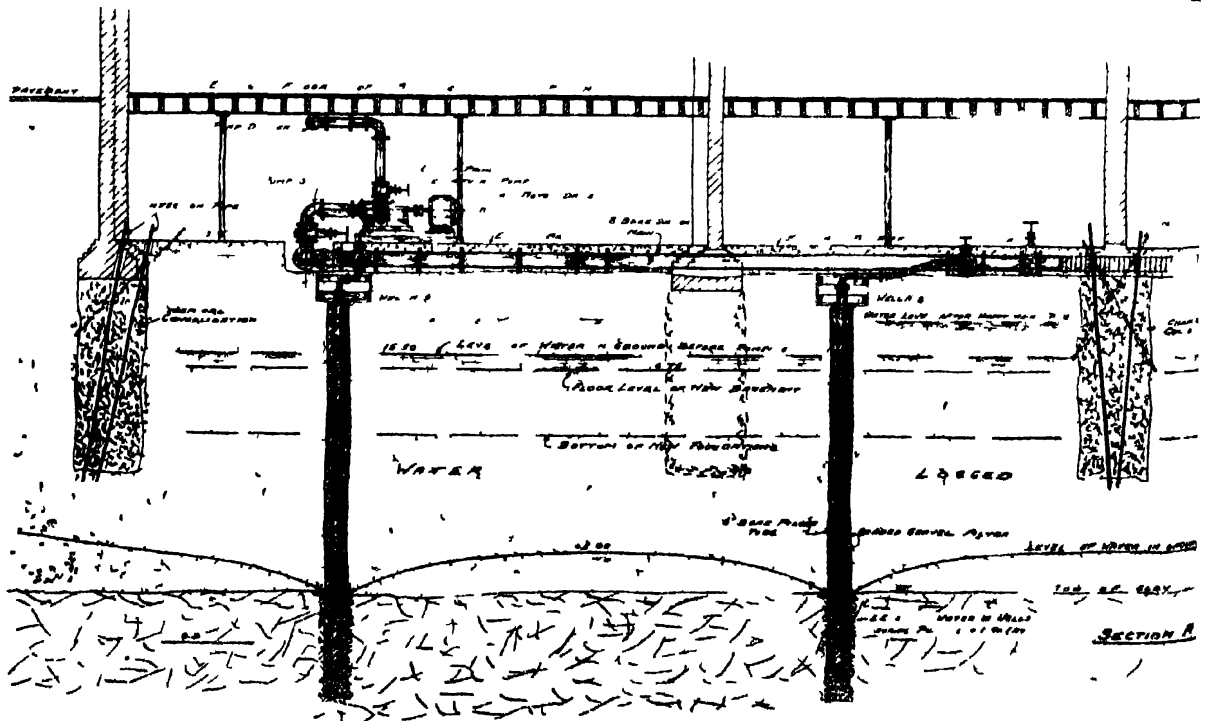
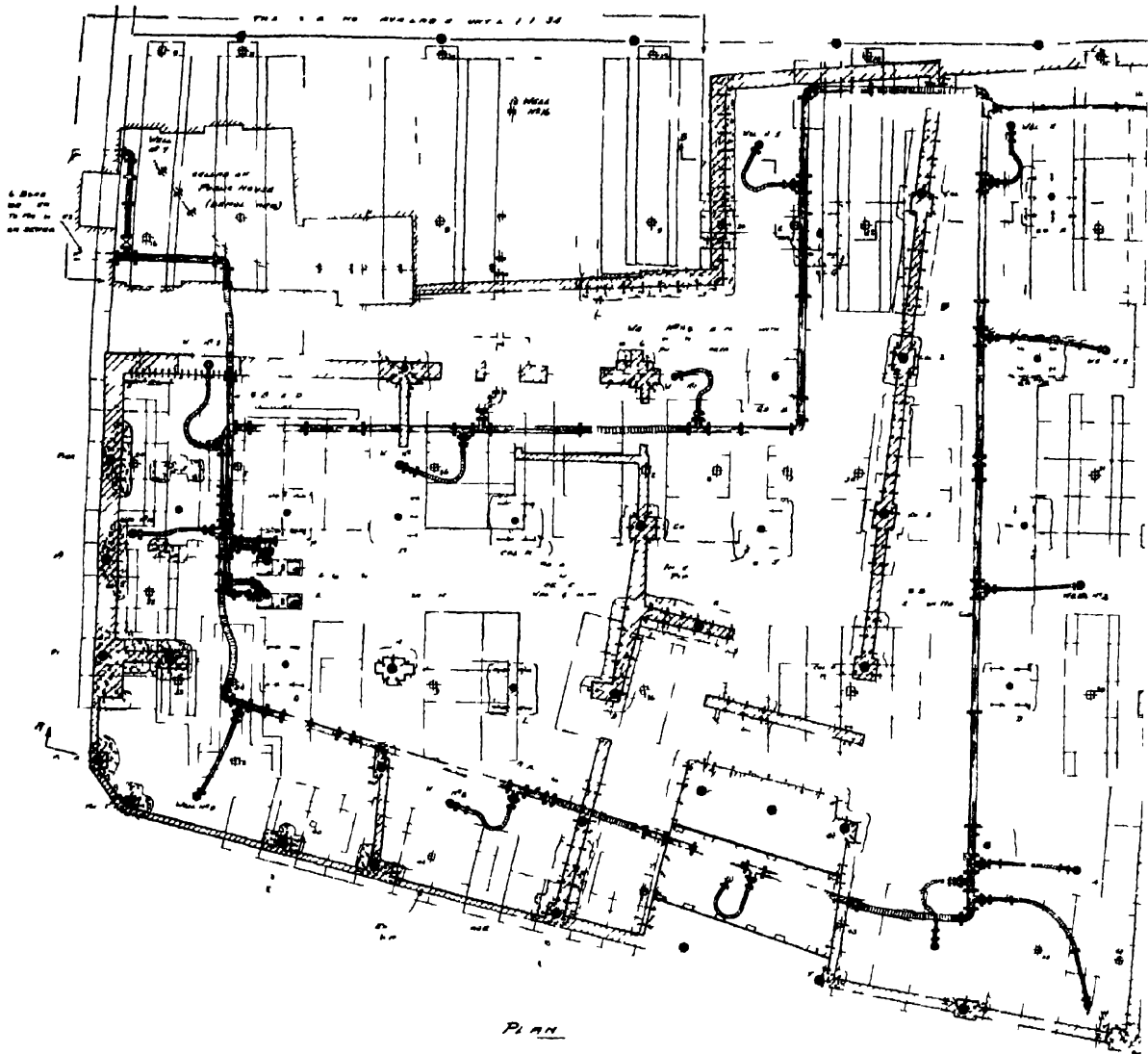
The scheme adopted for the reconstruction of this portion was dictated by economic considerations. The premises were in full occupation and it was necessary for the shopping floors to be undisturbed during the heavy Autumn and Christmas trade and for rebuilding to be carried out during the quieter Spring season to be in readiness for the Summer trade.

Mr. B. I. Hurst M.Inst.C.E., the consulting engineer for the rebuilding devised a method of completing the stanchion foundations and basement retaining wall for the new building without disturbing the foundations of the existing structure. This was made possible by working in the cellars of the existing premises. The foundations of these had been originally built at about $+16.0$ O.D. which was the water level in the ballast. In some cases the headroom was as little as 7 ft, the cellars being used only for storage purposes. In the new design the concrete raft is founded at $+10.75$ O.D. with basement floor level of $+14.75$ O.D., which provides a commodious lower ground floor space of considerable trade value.

It was found that if the retaining wall and foundations could be constructed in the cellars before demolition, four months would be saved in the time between demolition and rebuilding. For a rapidly-growing store this represents a saving of many thousands of pounds in trade, and prevents less obvious losses due to lack of service and disorganization. The programme allows for two or three floors to be returned to use within eleven weeks of closing.

Design of Foundations.

A foundation raft of concrete is being laid all over the site at $+10.75$ O.D., and the raft is tanked with three coats of asphalt laid on the bottom 6 in. of concrete. The basement floor level is $+14.75$ O.D. and in order to gain unobstructed space the bottom grillage joists are embedded in the bottom of the raft and chases left in the concrete to take the top tier of grillages. The retain-



Scrubbed Surfaces.

The simplest and cheapest method of exposing the aggregate is to wash off the cement with a stiff fibre brush and plenty of water as soon as the concrete is hard enough to permit this being done without loosening or picking out pieces of aggregate or pockets of sand. For brushing hardened concrete wire brushes with flat bristles, such as are used by butchers for scouring blocks, may be employed. These have the advantage that the bristles pass through holes in a movable plate and may be adjusted to give greater or less stiffness, as required according to the hardness of the concrete, by shortening or increasing the effective lengths of the bristles.

On the new Twickenham Bridge over the river Thames the bush hammered surfaces of the arches have a coarse texture, and reveal the pit ballast of which the concrete was made. The coping, the walls of the approaches, and the terminals to these walls and other details were pre-cast. While it was desired that the pre cast units should be generally similar to the bush hammered work the opportunity was taken of using a different texture so that the pre cast work would give relief to the bridge proper. The aggregates in the pre cast work consist of pit shingle and sand of the same general colour as the aggregates used for the bridge but a different texture has been obtained by the use of $\frac{3}{4}$ -in shingle to contrast with the $\frac{3}{4}$ in ballast used for the bridge. A further difference has been obtained by scrubbing the faces of the pre-cast units, as this process results in a smoother texture than the bush-hammered bridge elevations. The slabs were scrubbed with a wire brush within 12 hours of casting the stone.

The cost of exposing the aggregate by brushing with water depends upon the hardness of the face when the work is undertaken and whether plane surfaces or work such as bridge balustrades is to be treated. From 2s to 5s per square yard may be taken as outside labour costs for this class of work.

If when the forms are stripped the face of the concrete is too hard to permit exposure of the aggregate by scrubbing with water, the cement may be removed with a solution of hydrochloric acid. A

solution of one part hydrochloric acid to six parts water should first be tried, and if this is ineffective the solution should be strengthened until copious applications of the solution and brushing with a stiff brush remove the cement with reasonably vigorous brushing. As the hardness of the cement depends upon the age of the concrete, the conditions under which it has been cured, and other factors, no definite proportions of acid and water can be laid down as being effective in all cases, it is best first to try a small patch with a weak solution and strengthen it as necessary. The effect obtained by the use of acid solution is similar to that obtained by scrubbing with water only. The solution disintegrates the cement so that it can be brushed off, and if a uniform surface is desired care must be taken to remove the cement to the same depth over the whole of the area being treated.

It is necessary thoroughly to wash from the concrete all traces of the solution after it has served its purpose or it will continue to act on the cement and to disintegrate the face of the concrete, resulting in pieces of aggregate becoming loose and falling away. An alkaline solution is sometimes brushed over the acid treated surface in order to neutralise any free acid remaining on the concrete and prevent further chemical action. Acid treatment is undesirable on reinforced work with small cover or low weight concrete, and acid must not be used if the aggregates are limestone. Care must be taken to prevent the hands coming in contact with the solution and it is advisable to wear rubber gloves when using it.

Acid treatment of concretes of different ages will result in different degrees of finish. Generally it is impossible to follow up the form stripping by acid treatment and the latter has to be executed at one time with consequent variations in the results unless the workmen scrub the older concrete more than the new with a view to obtaining uniform results. The only additional cost of the acid treatment compared with scrubbing with water is the cost of the acid.

Cement Retarders.

Retarders, which are in the form of liquid or jelly, are brushed over the forms before the concrete is placed and

retard the setting of the surface cement so that it may be easily brushed off. If the forms are removed within a week or so it is found that the surface cement is powdery and can be removed dry with a brush. Retarders are available of varying strengths which penetrate the face of the concrete to different depths so that pre-determined textures can be obtained.

Special Faces applied with Sliding Shutters.

To overcome the difficulty of rendering concrete work satisfactorily a sliding plate, fitted with angle iron strips riveted to one side to give the required thickness to the mortar surfacing layer, has been developed. This enables a good bond to be obtained between a surfacing of white or coloured concrete and a grey concrete backing. An example of the application of this method in engineering work occurred in the construction of the stepped downstream face of the Ryburn Dam. In this case a steel plate shutter 2 ft 6 in deep provided with lifting hooks was used to separate the facing concrete from the backing. Wooden spacers notched to fit over the top edge of the steel plate were used to retain the plates at the correct distance from the timber panel forms.

When a finish of fine concrete is to be applied integrally with a backing of coarser grey Portland cement concrete in reinforced work, it is important to make the thickness of the former sufficient to give ample room for the shutter to slide without fouling the reinforcing bars. A layer less than 1 in thick is difficult to place in this manner and calls for special care and supervision. While smaller thicknesses have been tried the ultimate cost due to increased labour charges has often exceeded the additional charge which would have arisen by using a thicker coating of the more expensive concrete. For work which is to be lightly tooled a thickness of 1 in is satisfactory if the surface concrete is carefully placed.

In using the sliding shutter the level of the fine facing concrete should be kept at least 2 in or 3 in above that of the backing so as to prevent the latter penetrating to the face. At the ends of the shutter the facing concrete should be filled into the formwork, extending around the ends of the steel plate into the grey

concrete behind. This is the only satisfactory method of keeping the two separated on the surface. It requires more fine concrete than the net quantities taken off the drawings show, and an ample allowance for this "waste" must be made when tendering.

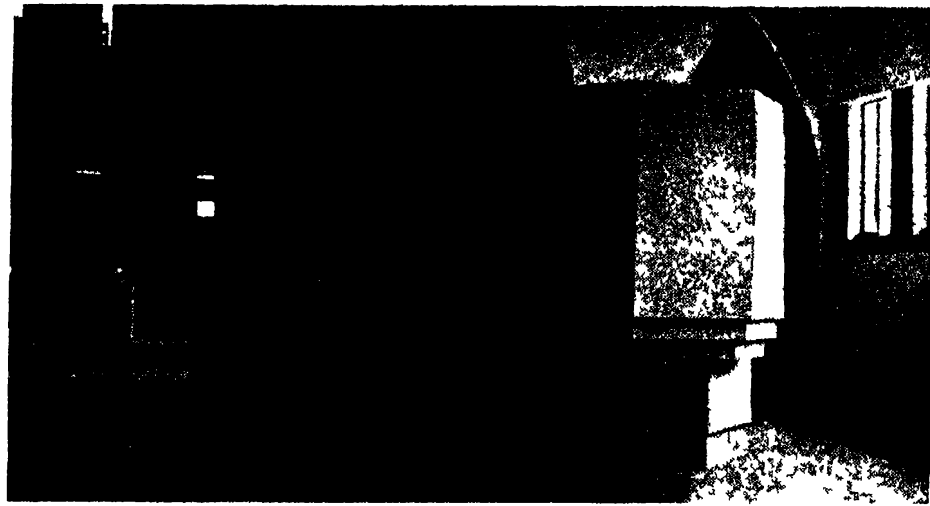
Experience has shown that surfacing can be deposited in 12-in depths before lifting the steel separating shutter. The plates must be carefully washed and scraped after use to ensure a good finish.

The consistency to which the white facing concrete is to be mixed is best determined by trial. The considerations governing the consistency are practical rather than theoretical, if the mix is very wet its capacity for preventing penetration of the grey concrete backing is reduced and patches of dark concrete may be formed on the finished surface. On the other hand the use of an excessively dry mix is almost bound to be accompanied by fractures in the surface layer when the shutter is lifted. These also cause penetration by the grey concrete, and necessitate ultimate cutting out and patching of the surface. The latter must be avoided at all costs if the final appearance of the surface is not to be marred, since it is almost impossible to make an invisible patch on concrete work.

From the theoretical point of view there would appear to be a disadvantage in making the facing and backing concretes of different consistencies, as this would result in unequal shrinkage of the two layers during the period of hardening. This, however, is not a very important point, and practical experience has proved that satisfactory results can be obtained with different consistencies in the two layers.

For this class of work provision must be made in the design to facilitate placing the two layers of concrete. It is, for example, better to place the horizontal bars in the wall behind rather than in front of the vertical reinforcement so as to prevent them fouling the steel plates when these are being lifted. The vertical bars also act better as a guide to keep the thickness of the facing concrete uniform.

Some methods of detailing and fixing the vertical reinforcement obstruct the travelling plate and should be avoided. For example, a structure which comprises



*Tempered Presdwood forms provided the beautiful finish to this concrete Pulpit with Lectern in correspond at Bishop Andrews Church St Helier Surrey
Architect: Giddes Hyslop Esq A.R.B.A.
Contractors: H. M. Nowell Ltd.*

To be continued——

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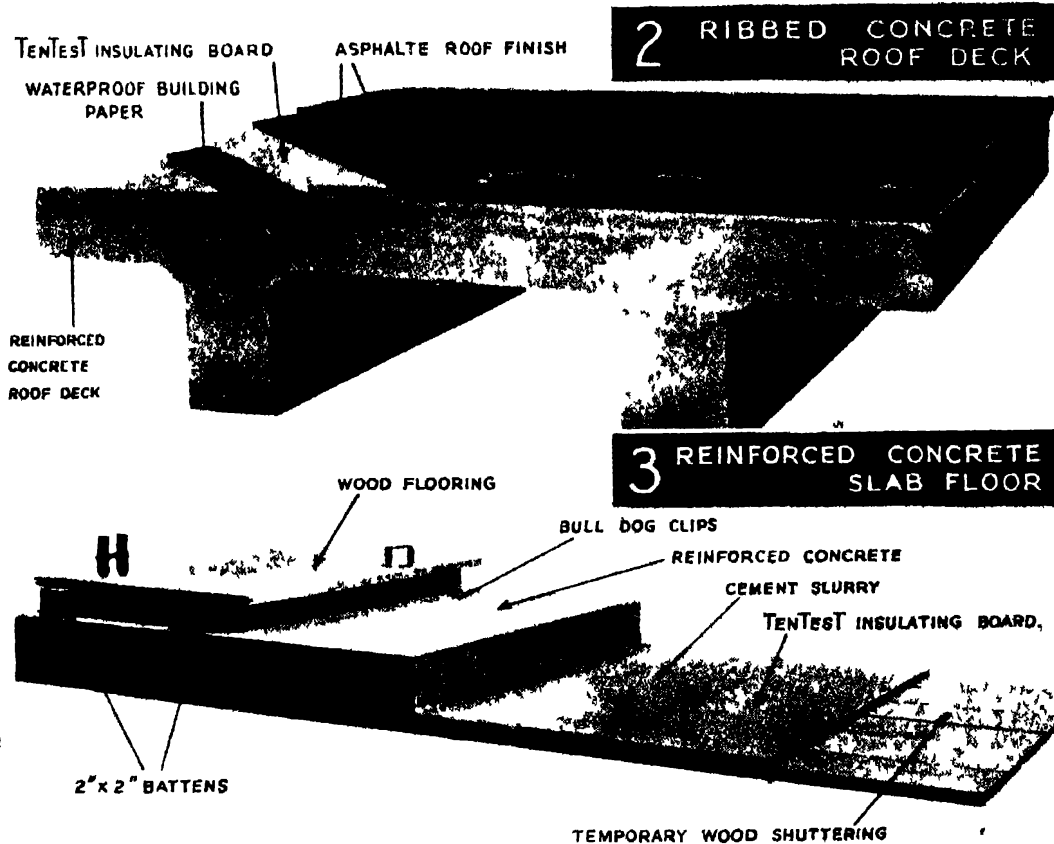


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a vertical wall and a sloping slab roof should not have the continuity bars bent over before the sliding shutter reaches the top of the vertical wall. If this is done these bars prevent the shutter being raised to finish the vertical surface. The bars may be bent when the latter is completed and the concrete has hardened, this being a case when the standard practice by which bars are not bent in position may be abandoned.

Experience shows that success in obtaining a good face depends on the accurate grading and even mixing of the facing concrete, on placing the concrete in a fairly dry state so that the larger stones do not tend to sink to the bottom on even tamping in small lifts and on removing any scum which may rise to the top at the completion of each lift or particularly at the end of each day's work. The first layer after a stop should be placed slightly wetter and very thoroughly tamped.

The face centring should be removed as early as possible and the concrete treated with a wire brush to give the

desired texture. If any blemishes occur they should be filled, immediately after the concrete is brushed, with the screenings from the dry concrete according to the gauge required.

This method was used at Atcham Bridge built over the river Severn for the Shropshire County Council, the whole of the elevations being finished to resemble the local Grimshill stone. The special face of a depth of 2 in., is composed of 3 parts $\frac{1}{2}$ in. to $\frac{1}{4}$ in. Pontesbury quartzite, 1 $\frac{1}{2}$ parts Criggion stone, $\frac{1}{4}$ in. to dust and Leighton Buzzard sand in equal proportions and 1 part coloured Portland cement. The surface was bush hammered with pneumatic hammers at periods varying from one month to six months after the concrete was placed. The cost of labour was 1s. 1d. an hour, and the contract price for bush hammering was 3s. per square yard for all surfaces. Since that date the contract price for bush hammering on a smaller bridge in the same county was 2s. 3d. a square yard for flat surfaces and 3s. a square yard for mouldings and copings.

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In a recent case where a steel shutter was used on balcony walls to separate a white Portland cement concrete facing from a grey concrete backing, the front and back forms for the wall consisted of waterproof plywood panels $\frac{5}{8}$ in thick with stiffened edges some of which were used as many as fifty times before being scrapped.

Pre-Cast Facing Slabs.

If pre cast concrete slabs are used to face a reinforced concrete structure economy can often be effected by using the slabs as shuttering. An outstanding example of this method of construction is the Dorchester Hotel, London, where cream coloured slabs were used. The external walls are 11 in thick overall, comprising a 2 in facing slab, 7 in of reinforced concrete, and a 2 in lining of cork insulation. The slabs and cork lining were used as external and internal shuttering respectively. The slabs are 2 in thick, comprising an outer face $\frac{1}{2}$ in thick of a mixture of crushed cream-coloured marble and cream coloured

Portland cement mixed in the proportions of 1 part cement to $2\frac{1}{2}$ parts aggregate and $1\frac{1}{2}$ in of concrete made with ordinary Portland cement and shingle. Reinforcing wires projecting from the backs of the slabs hold the slabs firmly to the poured concrete wall. The slabs were made face down in steel moulds and afterwards polished with carborundum disks. After three years of exposure to the London atmosphere the mortar joints between the slabs showed signs of discoloration and some of the slabs had lost a little of their original brightness. It was therefore decided to clean the whole of the elevations of the building with electrically-operated polishing machines with fine grade disks, similar to those used for cleaning down the slabs before the hotel was opened. With a view to preventing future discoloration, after the walls were cleaned down they were coated with cellulose.

In constructing the side walls of the open air swimming pool at Epsom for the Royal Automobile Club, pre cast concrete slabs were used as shuttering

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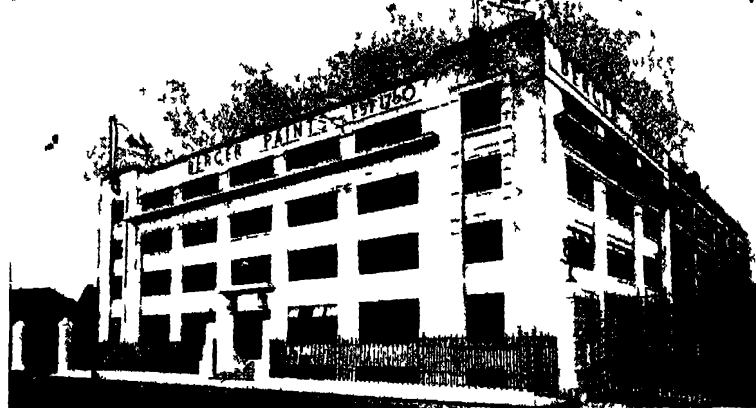
3 parts crushed Crowborough stone through a 20 x 20 sieve, 1 part of the following mixture 60% white Portland cement, 40% grey Portland cement, 4% CARDWELL'S YELLOW OCHRE

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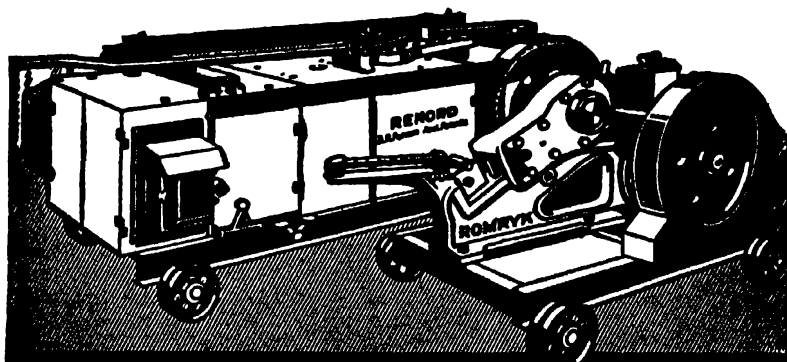
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for the inner face of the reinforced concrete wall and were supported by a light timber framework with raking bracing. In this way the cost of sheeting for these forms was saved. On the outer side of the wall the shuttering consisted of timber panels strutted off the side of the excavation. When fixed in position the slabs were polished by electrically driven machines. The slabs were manufactured on the site and are 2 in. thick and comprise 1½ in. of ordinary concrete backing with a ½ in. facing of a special mixture of calcined flint aggregate and coloured Portland cement supplied ready mixed by the cement manufacturers. In order to reduce the amount of labour required in polishing, the slabs were rubbed down with newspaper before the concrete hardened appreciably. It was found that this was superior to the ordinary practice of rubbing down with sickle.

This type of precast slab is being increasingly used for lining swimming baths due to its economy compared with other materials. A recent estimate for an open air bath 100 ft. long by 30 ft.

wide included £1,376 for terrazzo, £1,852 for white glazed tiles and £2,746 for white glazed bricks.

Applied Colours.

Proprietary materials are now available in many shades which may be brushed on to concrete surfaces, and which have proved constant after many years' exposure. These colours are supplied in liquid form ready for use, and are applied with a brush. Plain stippled and other finishes are available. A notable example of this treatment is the chimneys of the new Bittersea Power Station which are finished in an attractive shade of buff. It is necessary thoroughly to clean the concrete surfaces before the colour is applied, steam jets sometimes being used for this purpose. On the chimneys referred to a priming coat and two finishing coats were applied.

On the bridges recently completed at Peterborough the surfaces were colour-sprayed by means of a cement gun, using a mixture of Ketton Oolites and cream coloured cement and a consider-

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able amount of experimental work has been undertaken recently on the use of the cement gun for the application of coloured cement sand mixtures. When commercial sands and grey cement are used for gunite renderings on industrial structures a cloudy or patchy surface often results due either to variations in the colour of the sand, to slight variations in the amount of water used in the mix, to the sand which rebounds from the surface being blown on to parts already rendered, or to matching one day's work on to work done the previous day. With careful selection of materials and careful workmanship this defect can be avoided, and, as on the Peterborough bridges, coloured mortar of a thickness just sufficient to cover the concrete can be applied with a uniform result. On fair size contracts the cost of spraying a white or toned facing by the cement gun should not exceed 3d. or 4d. per square foot, including plant, materials and scaffolding up to 30 ft. or so in height. The work can be done very quickly, and only one coat is necessary.

Coloured Cements.

The introduction of coloured Portland cements and ready mixed coloured

cements and aggregates during the past few years has considerably simplified the production of coloured concrete. But if the colour is to be added by the user care must be taken to ensure its even distribution if a patchy result is to be avoided. The cement and colour must be mixed in the dry state, preferably in a ball mill. Failing this, the cement and colours should be passed through a fine sieve until they are uniformly distributed. Only the best metallic oxides should be used, and as these have a weakening effect on the concrete they should not be used in proportions exceeding 15 per cent of the cement content. A smaller proportion, say, 10 per cent, is desirable, and it will often be found that a desired shade can be obtained by the use of smaller proportions of darker shades than if one shade only were used throughout. When matching colours it is advisable to mix a small batch, keeping a careful record of the quantities of materials and colour used, and make a trial slab so that the shade may be noted after the slab has dried out. If this is not the exact shade desired, the proportion of colour should be varied in other trial slabs until the desired shade is obtained after the slabs have dried out.

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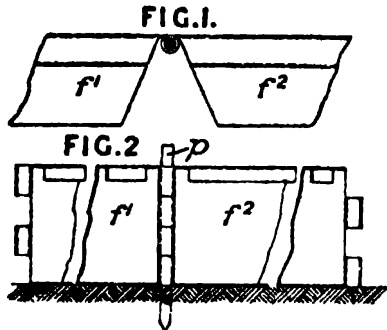
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Recent Patents Relating to Concrete.

Edge Moulds for Concrete Road Construction.

395,442.—Grant, A, Arnwood, Kingscroft Road, Leatherhead Feb 20, 1932

Panel moulds (f^1 f^2), in short lengths, are provided with interfitting hinge portions on their ends through which an



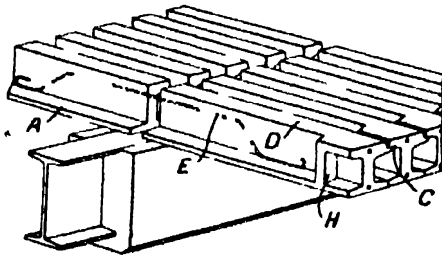
EDGE Moulds FOR CONCRETE ROADS

anchoring pin (p) passes to position and secure the moulds. The hinge portions are so shaped that a continuous moulding surface is presented whatever the angle the moulds are disposed at to each other in forming curved edges.

Floors and Roofs

398,506.—Rapid Floor Co Ltd 59 New Oxford Street London (Schlagintweit, F, 7, Kronprinzstrasse, Baden Baden, Germany) March 11, 1932

Relates to a floor, roof, or like construction employing pre cast beams (A)



FLOORS AND ROOFS

placed side by side and consists in making provision for the negative bending moments which occur at supports or fixed ends in continuous constructions by omitting part or the whole of the upper flanges of the beams towards the ends of the spans in order to provide spaces into which reverse bending moment rein-

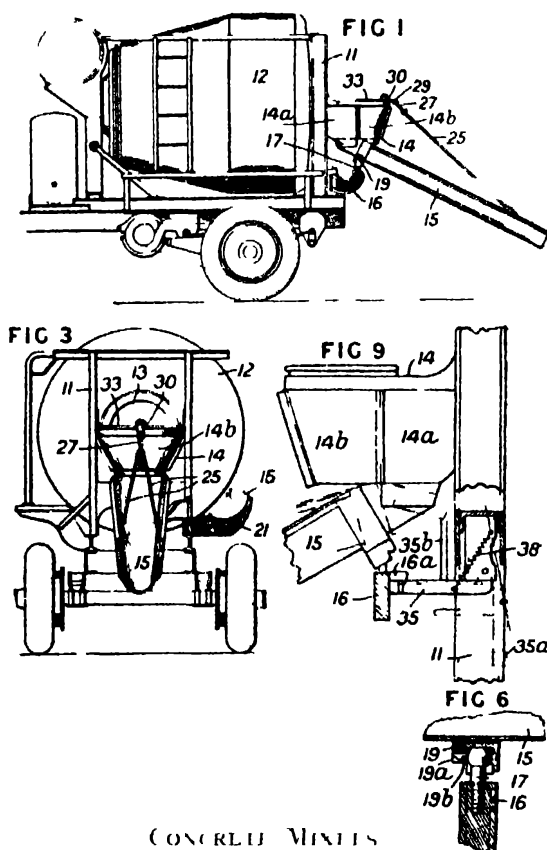
forcement (E) is placed and concrete cast. The beams may be of I section, as shown in the figure, with the upper flanges (C) omitted on one side of the beam along the portions (D). The concrete filling extends to the upper surface of the beams. Stop members (H) to restrict the passage of the concrete may be cast integrally with the beams or formed separately of concrete or timber. The length of each portion (D) should not be less than one sixth of the span. One flange of each I beam may be formed as a ledge to support the flange of the adjacent beam. In the case of beams of hollow box section, slots are formed in the top flanges for the insertion of the reinforcement and concrete.

Concrete Mixers.

400,342.—Potter, E, 3, Staple Inn, High Holborn London (Chain Belt Co, Milwaukee, U S A) Jan 22 1932

Means for distributing concrete from a concrete mixer (12) consists of a delivery chute (15) adapted to receive concrete from a discharge opening in the mixer and arranged so that it is rotatable into an inoperative position wherein it lies substantially parallel to the longitudinal axis of the mixer and adjacent thereto as shown in dotted lines in Fig 3. The chute (15) is mounted at its receiving end on a universal pivot (17) on an arm (16) which is pivotally connected to the frame so as to have substantially horizontal swinging movement. The free end of the chute is supported by a rod or cable (25) connected at its upper end to a strap (27) having a series of apertures for the reception of a bolt (29) carried by a trolley (30) adapted to run on a track (33) on a delivery hopper (14). This hopper (14) is mounted adjacent the discharge opening (13) in the mixer and above the receiving end of the chute (15), and comprises side plates (14a) and, if necessary, a curved end member (14b). The track (33) consists of a pipe welded or otherwise secured to the upper edge of the hopper. The socket member (19) of the universal pivot (17) is cut away as at 19a, Fig 6, to permit the chute to be tilted laterally when in the inoperative position, and a bolt or key (19b)

is provided to prevent unintentional separation of the parts of the joint. In the inoperative position the free end of the chute is supported by an arm (21) rigidly mounted on the frame (11) as shown in Fig. 3. One or more stop members (35) are provided to prevent the arm (16) and the receiving end of the chute from coming into contact with the drum (12) or the frame members (11). The stop member (35) consists of an arm pivoted to the frame (11) and



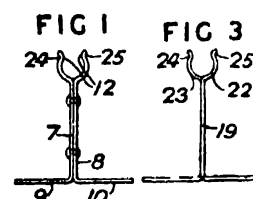
CONCRETE MIXERS

held in a horizontal position by a spring (38). When the receiving end of the chute (15) is in proper position beneath the hopper (14) latch members (16a, 35a) carried by the arms 16, 35 respectively engage in order to prevent the receiving end of the chute being displaced relatively to the hopper (14). The arm (35) is provided with an upstanding rod (35b) adapted to be engaged by the receiving end of the chute as it swings downwardly so as to depress the arm (35) against the action of the spring (38) and release the engagement between the latch members (16a, 35a). The arm (16) may then be swung about its pivot

Reinforced Concrete.

400,948.—Dunker, J. H., Mole House, 1 Sher Road, Walton on Thames. May 2, 1932.

A supporting member for positioning reinforcing mats or bars within concrete structures comprises a base member having an upstanding projection the upper end of which is provided with a fork adapted to engage by snap action the portion of the mesh or bar to be supported. The fork is adapted to be opened out in the process of inserting the mesh or bar but tends to return to its original position after the mesh or bar has been inserted. As shown in Fig. 1 the member comprises a pair of strip metal members (7, 8) having their lower ends (9, 10) bent outwardly to provide a base member and with their main parts connected by rivets, bolts, welding, etc. The upper



REINFORCED CONCRETE

ends of the strips are opened out to provide the fork (12). In a modification the support is made from a single strip (19) (Fig. 3) having its upper end divided to provide three parts of which two parts (22) are bent in an opposite direction to the other part (23) in order to provide a fork. The upper edges of the fork are bent back as at 24, 25 to facilitate entry of the reinforcement into the fork, and also to permit where desired a securing wire to be wrapped around the ends of the fork. The securing wire may alternatively be passed through holes in the supporting member or completely around such member. The forked members may engage only partially a reinforcing bar.

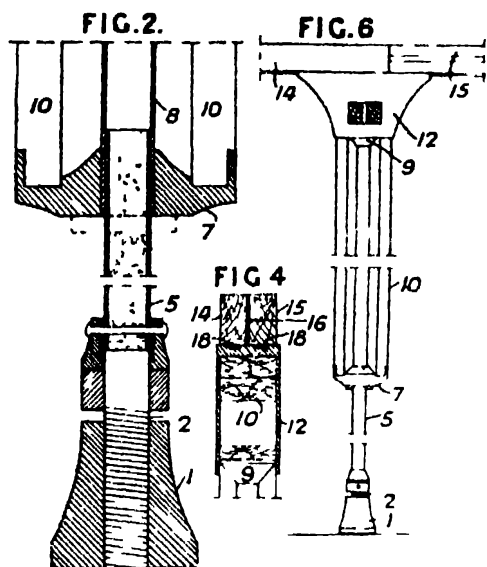
Shuttering Supports.

403,279.—Valentine, C. R., P.O. Box 451, Vepery, Madras and Greaves, J. H., P.O. Box 63, Bangalore, Mysore, India. April 24, 1933.

A support for shuttering of the type comprising adjustable horizontals and a telescopic prop having rough and fine adjustment has a frame carried by shoes slidably adjusted on a tubular upright

which has a fine adjustment in relation to a metal base. The fine adjustment is provided by a screw (2) in the base (1), the screw supporting a concrete filled

reversed, the apertures (b, g) coincide, and cement grout, supplied through a



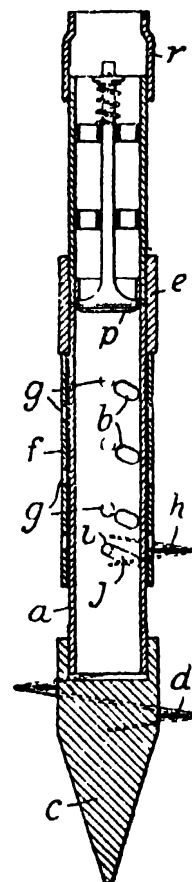
SHUTTERING SUPPORTS

tube (5) on which slides a sleeve (8) having upper and lower shoes (9, 7) respectively carrying a hardwood frame (10) and cap (12). The cap carries the adjustable horizontals (14, 15) at its upper end by means of dovetail feathers (18), the horizontals also having a dovetailed sliding connection (16) to each other.

Cement Guns.

405,767.—Leeds, K. I. N., Werrington House, Werrington Peterborough. Nov. 10, 1932.

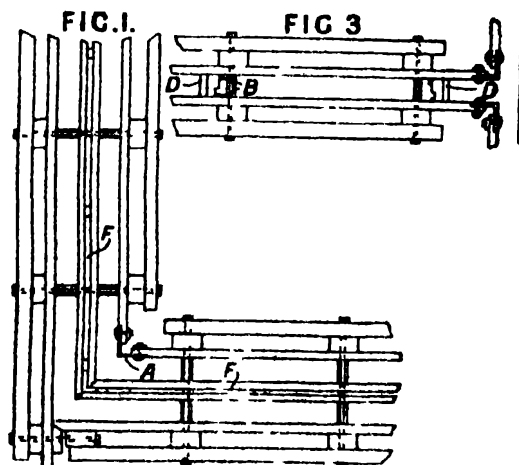
A device for introducing cement grout into substrata for strengthening them comprises a length (a) of tube with four equi-angularly spaced series of oblique slots (b), a shoe (c), and, at its upper end, a sleeve (e). Sliding on the tube (a) is a sleeve (f) with a series of round apertures (g), corresponding with the slots (b) and a helical fin (h). A grub screw (i) on the sleeve (f) engages in a slot (j) in the tube (a), the length of the slot being equal to that of a slot (b) plus twice the diameter of the screw (i), the arrangement being such that when the screw (i) abuts against the upper end of the slot (j) the fin (h) lies in the same helix as the fin (d) on the shoe (c). The device being screwed into the ground, the fin (h) causes the tube (f) to rise and to close the apertures (b). When the rotation is



coupling (i) and a spring supported valve (p) can be pumped through them.

Moulding Walls in Situ.

406,954.—Long, J. W. 141, Dover Road, and Whaling, J. L., 15, St John's Road, both in Ipswich. Oct. 28, 1933.



MOULDING WALLS IN SITU

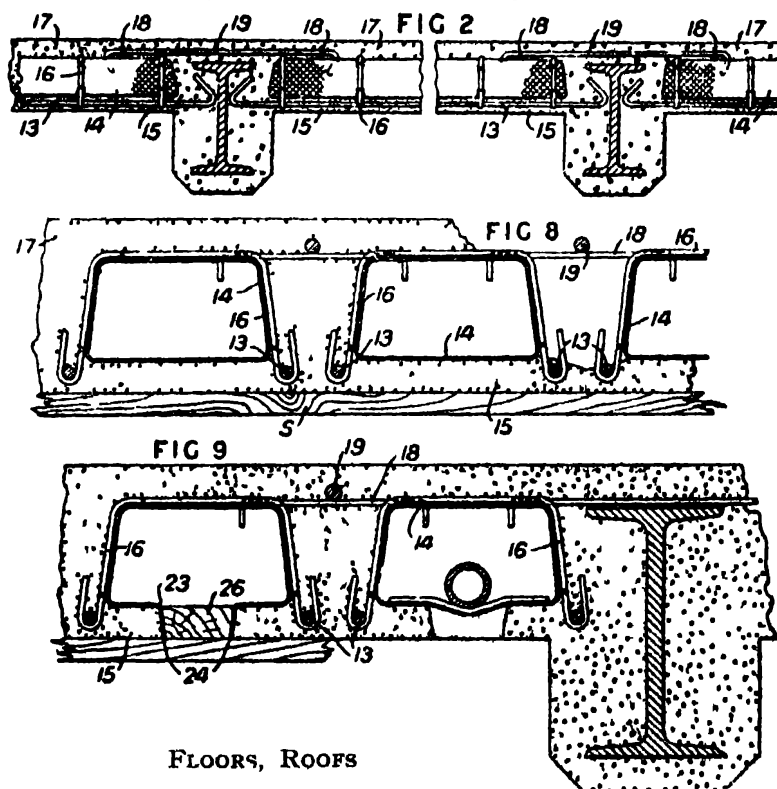
For moulding walls in situ wooden shutters are connected at the internal angles by angle connectors (A) which enter rabbets in the ends of the shutters and are bolted thereto. The corners of the outer shutters are bolted together as shown. Internal core shuttering (I) is provided to form hollow walls when required. The doors and window frames are positioned on the shutters by fillets (B Fig 3) and dovetail blocks (D) are provided on their rear edges to lock them to the concrete. The rear edges of the frames may be provided with dovetail grooves instead of blocks.

Floors, Roofs.

407,579. Indented Bar & Concrete Engineering Co Ltd and Vawdrey R W, Vincent House Vincent Square London June 15 1933

A method of constructing hollow reinforced concrete floors and plane surface

for supporting the main longitudinal reinforcing rods (13). In casting the floor, a core (14) provided with the stirrups (16) and reinforcing rods (13) is positioned on a layer (15) of concrete disposed on the shuttering (5) and then further concrete (17) is applied, as shown in Fig 8, to surround the core. The other cores (14) are then successively positioned and embedded in concrete. Adjacent cores may be tied together by bars (18) and additional upper reinforcing rods (19) may also be provided. The main reinforcing rods (13) extend from joist to joist as shown in Fig 2 and the cores (14) terminate at a short distance from the joists. The cores are shortened or interrupted at places where openings are provided in the floor. To enable the cores to house pipes electric conduits etc they may be provided with longitudinal openings (23) formed by bending downwardly the longitudinal opposed edges of the metal to form flanges (24) as shown in Fig 9 and such openings are filled during casting.



FLOORS, ROOFS

roofs consists in placing tubular cores (14) of expanded metal metal mesh metal lattice, or like openwork metal in position in the structure and totally and permanently embedding it in concrete to form a hollow reinforced concrete slab the upper and lower faces of which are composed entirely of concrete. Stirrups (16) are used to secure the cores (14)

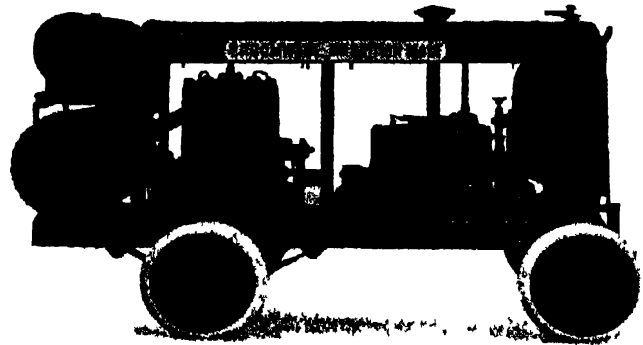
with wood members (26). The pipes, etc., are inserted after the floor has been cast, and the flanges (24) are then bent upwardly and the opening (23) is filled with concrete. If desired, selected portions only of a floor may be cast in the manner described, the remaining portion being formed of pre cast hollow reinforced concrete slabs.

January, 1935.



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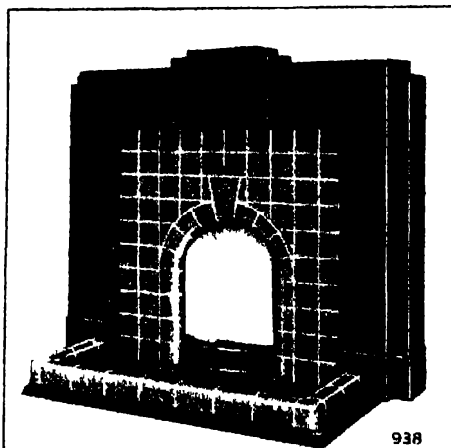
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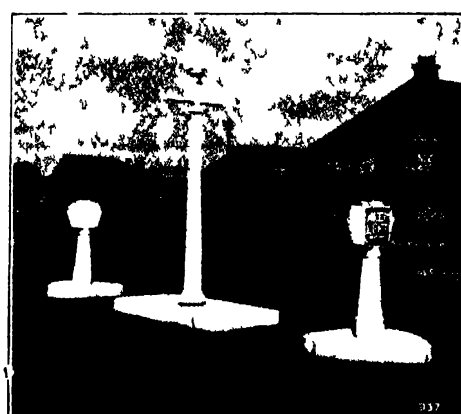
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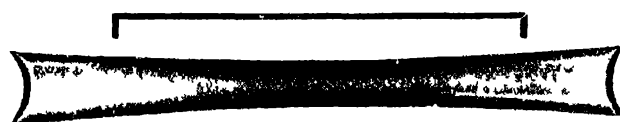
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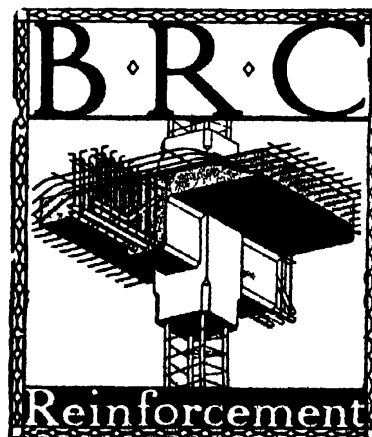
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DATA FOR PRICING REINFORCED CONCRETE.*

MATERIALS DELIVERED 4 MILES FROM CHARING CROSS				s	d
Best Washed Sand		per yard		8	3
Clean Shingle, $\frac{3}{4}$ in mesh		"		7	3
" " $\frac{1}{2}$ in mesh		"		9	0
Thames ballast		"		7	3
Broken brick ($\frac{1}{2}$ in)		"		11	3
Best British Portland Cement (delivered London area in 4 ton loads and upwards) in jute sacks or non returnable paper bags per ton 40s, 1 ton to 3 ton 19 cwt loads, 12s per ton, jute sacks charged 1s 6d each and allowed 1s 6d each if returned in good condition within 3 months					
Rapid-Hardening Portland Cement delivered London 6s per ton extra					
'Colorcrete' red and buff rapid hardening Portland Cement delivered London in non returnable paper bags per ton 66s					
'Cullamix' Coloured Portland cement and aggregate delivered London area from £5 per ton upwards					
'Snowcrete' White Portland Cement, £5 15s per ton delivered London including non returnable paper bags					
'Super Cement' Waterproof Cement	(paper bags free)	per ton	70s		
BOARDING FOR SHUTTERING.—					
1 in	Sawn				
1 $\frac{1}{2}$ in	per square	21s 0d	25s 0d		
1 $\frac{1}{2}$ in	"	26s 0d	30s 6d		
	"	31s 0d	36s 0d		
SAWN TIMBER FOR STRUTS AND SUPPORTS					
3 in by 4 in					
3 in by 6 in and 3 in by 7 in					
	from £10 per standard				
	£15				
MILD STEEL RODS FOR REINFORCEMENT —				s	d
$\frac{1}{8}$ in to $\frac{1}{4}$ in Rounds	per cwt			8	6
$\frac{3}{8}$ in to $\frac{1}{2}$ in Rounds	"			8	9
$\frac{5}{8}$ in Rounds	"			9	0
$\frac{3}{4}$ in Rounds	"			10	0
Breeze Slabs per yd super 2 in 1/6, 2 $\frac{1}{2}$ in, 1/8, 3 in, 2/4, 4 in 2/4					
MATERIAL AND LABOUR INCLUDING 10 PER CENT PROFIT					
(Based on Contracts up to £2,000)					
PORTLAND CEMENT CONCRETE WELL MIXED IN PROPORTION 1 2 4 —				s	d
Do do in foundations	per foot cube			1	
Do do in columns	"			1	4
Do do in beams	"			1	6
Do do in floor slabs 4 in thick	per yard super			3	8
Do do in floor slabs 5 in thick	"			4	7
Do do in floor slabs 6 in thick	"			5	6
Do do in floor slabs 7 in thick	"			6	5
Do do in walls 6 in thick	"			5	6
Add for hoisting 3s 6d per yard cube above ground floor level Add for rapid hardening Portland Cement 2s per yard cube)					
STEEL REINFORCEMENT INCLUDING CUTTING, BENDING AND PUTTING INTO POSITION AND SECURING WITH STOUT BINDING WIRE—					
From $\frac{1}{8}$ in to $\frac{1}{4}$ in	per cwt			15	0
" $\frac{3}{8}$ in to $\frac{1}{2}$ in	"			14	0
" $\frac{5}{8}$ in to $\frac{3}{4}$ in	"			13	0
LXTRA LABOUR TO BENDS in $\frac{1}{8}$ in rods $\frac{1}{2}d$, $\frac{3}{8}$ in rods $1d$, $\frac{1}{2}$ in rods $1\frac{1}{2}d$, $\frac{5}{8}$ in rods $1\frac{1}{2}d$, $\frac{3}{4}$ in rods $1\frac{1}{2}d$, $\frac{7}{8}$ in rods $2d$, 1 in rods $2\frac{1}{2}d$, $1\frac{1}{8}$ in rods $3d$, $1\frac{1}{4}$ in rods $3\frac{1}{2}d$, $1\frac{1}{2}$ in rods $4\frac{1}{2}d$, $1\frac{3}{4}$ in rods $6d$					
EXTRA LABOUR TO HOOK BENDS $\frac{1}{8}$ in, $1d$, $\frac{3}{8}$ in, $2d$, $\frac{1}{2}$ in, $2\frac{1}{2}d$, $\frac{5}{8}$ in, $3d$, $\frac{3}{4}$ in, $3\frac{1}{2}d$, $\frac{7}{8}$ in $4d$, 1 in, $4\frac{1}{2}d$, $1\frac{1}{8}$ in, $6d$, $1\frac{1}{4}$ in, $7d$					
SHUTTERING—				s	d
Shuttering and Supports for Concrete Walls (both sides measured) per square				40	0
Centering to Soffits of Reinforced Concrete Floors and Strutting, average 10 ft high	per square			42	0
Do do in small quantities	per ft super			0	6
Shuttering and Supports to Stanchions average 18 in by 18 in	per ft super			0	6
Do do as last, in narrow widths	"			0	7 $\frac{1}{2}$
Do do to sides and soffits of beams, average 9 in by 12 in	"			0	7 $\frac{1}{2}$
Do do as last, in narrow widths	"			0	8 $\frac{1}{2}$
Raking, cutting, and waste to shuttering	per ft run			0	3
Labour, splay on ditto	"			0	2
Small angle fillets fixed to internal angles of shuttering to form chamfer	"			0	3
WAGES.—The rates of wages on which the above prices are based are —Carpenters and joiners, 1/7 per hour; Carpenters working on old shuttering, 1/8, Labourers on building works, 1/2 $\frac{1}{2}$; Men on mixers and hoists, 1/3 $\frac{1}{2}$; Bar-benders, 1/3 $\frac{1}{2}$					

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Prospective New Concrete Work.

AYTHSBURY Swimming Pool—The T C is to construct in open air swimming pool at the Vale at an estimated cost of £6 167

BARNSTAPLE Sewerage and Sewage Disposal The R D C has applied for sanction to a loan of £5 000 for sewerage and sewage disposal works at Combe Martin

BERWICK Swimming Pool etc The T C is considering the construction of a swimming pool approach road and sewerage works at a cost of £20 357

BILLINGHAM Bridge The Durham C C is to construct a bridge over Belasis Lane level crossing at Billingham

BRIGHTON Bridge The T C proposes to reconstruct Dyke road Drive railway bridge at an estimated cost of £20 000

CAISIOR Water Supply The R D C is to construct water supply works at an estimated cost of £23 000

CAMBRIDGE Swimming Pool The T C is recommended to construct a swimming pool at Coldham Common at an estimated cost of £3 460

COLERAINE Harbour Works—The Harbour Board proposes to extend the harbour at an estimated cost of £3 000

DORKING Sewerage and Sewage Disposal—The U D C has applied for sanction to a loan of £5 600 for sewerage and sewage disposal works

FAST DURHAM Sewage Disposal The U D C has approved a proposal for reconstructing the sewage disposal works at an estimated cost of £6 400

EDINBURGH Chimney—The T C is considering the erection of a chimney at the Portobello power station at an estimated cost of £60 000

ELY Water Supply The R D C is to construct water supply works at an estimated cost of £37 000

GLASGOW Roads—The T C has been recommended to reconstruct Springborn Gardens at an estimated cost of £38,000

ISLE OF AXHOJML (Lincs) Water Supply—The R D C is to construct water supply works at an estimated cost of £52,000

LANARK Bridge—The C C has approved the construction of a bridge,

CONCRETE RETAINING WALLS BASEMENTS & FLOORS WATERPROOFED

The addition of No 1 Metallic Liquid to gauging water will make mass concrete, cement renderings and floor toppings waterproof and dustless Fully guaranteed

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30 Denman St., London Bridge, S E 1

Millhugh Larkhall at an estimated cost of £4 200

LONDON Swimming Pool—The T C C has been recommended to approve an estimate of £25 000 for the construction of an open air swimming pool at Victoria Park

MARGATE Reservoir—The T C has received sanction to a loan of £26 342 for the construction of a 5 000,000 gall reservoir at Hete

MIDDLESEX Viaduct etc—The C C is considering the completion of Western Avenue including a viaduct about a mile long at an estimated cost of £351,695

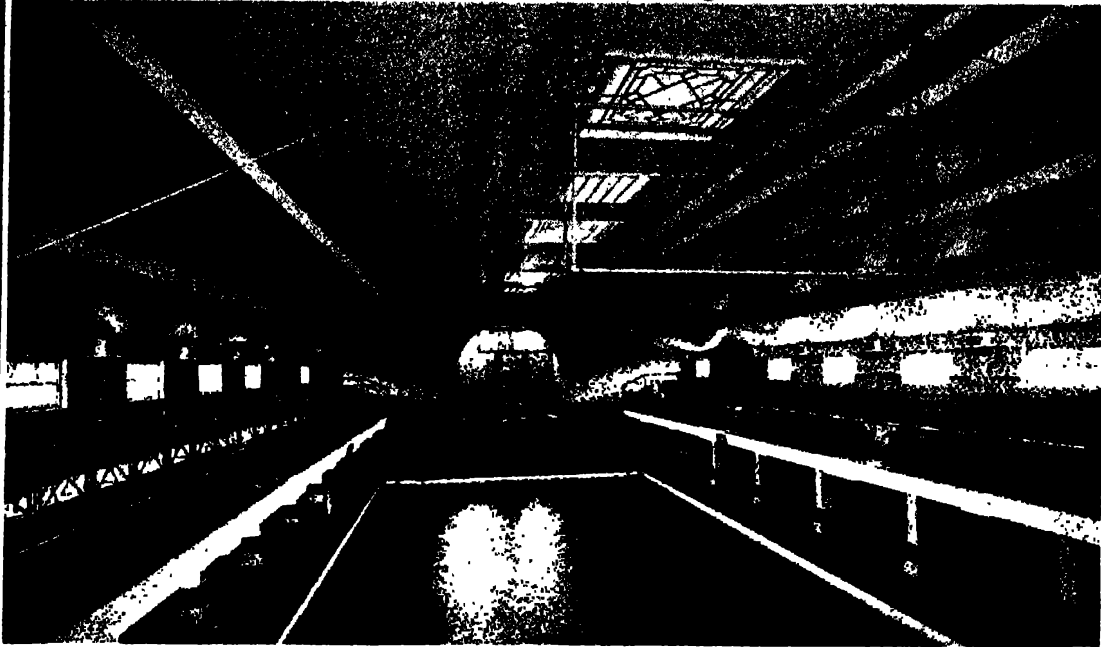
MIDDLESEX Bridge Widening—The C C in conjunction with the G W R is to widen the bridge over the railway at Station road Hayes by means of steel girders carrying a reinforced concrete deck

PENICUICK Reservoir—The B C has approved the construction of a reservoir at Quarrel Burn at an estimated cost of £13 500

PORTSMOUTH Road Reconstruction—The T C has agreed to construct a road from the Guildhall to Fratton, at an estimated cost of £206,100

RAMSGATE Bathing Pool—The T C is considering the construction of a bathing pool near Winterstroke Gardens, at an estimated cost of £35,000

January, 1935.



NEW SWIMMING BATH, MITCHAM.
Waterproofed throughout with "CALLENDRITE."
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instead of costing money. Every manufacturer can procure the hastening of the construction of a factory by seeing that "Ferrocrete" the rapid hardening Portland Cement is specified for all the concrete work involved. In 3 to 4 days concrete made with "Ferrocrete" attains the same strength as concrete made with ordinary Portland Cement in 28 days.

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RAMSGATE — *Sea Defence Bathing Pool etc* — The T C is considering the construction of an undercliff promenade sea defence works bathing pool and shelters at an estimated cost of £60 000

SALFORD — *Water Supply* — The R D C is to construct water supply works in Hensham and district at an estimated cost of £16 500

SEAFORD — *Sewerage and Pumping Station* — The U D C is to construct drainage works and a pumping station at an estimated cost of approximately £16 000

SHIELLY — *Sewerage* — The U D C has applied for sanction to a loan of £8 000 for the construction of sewerage works

SHIELLY — *Water Supply* — The U D C has applied for sanction to borrow £4 130 for water supply works

STIRLING — *Bridge* — The C C is considering the widening of the bridge over the river Allan at an estimated cost of £11 000 or alternatively the construction of a new bridge at estimated costs of £38 000 and £41 000

SURREY — *Bridges* — The C C has recommended the reconstruction of 16 bridges in the county

SWANCOMB — *Swimming Bath* — The U D C proposes to construct a swimming bath at an estimated cost of £6 000

TICSMOUTH — *Concrete Shelter* — The U D C propose to construct a reinforced concrete shelter on East Cliff terrace

TUNSTON — *Water Supply* — The R D C has applied for sanction to a loan of £4 130 for water supply works

WAKEFIELD — *Swimming Baths* — Plans are being prepared for the construction of swimming baths at an estimated cost of £25 000

WALLINGFORD — *Bathing Pool* — The T C has received sanction to a loan of £1 075 for the construction of a bathing pool

WALTON — *Bridge* — The U D C proposes to reconstruct the bridge over the river Wandle

WARMLEY (GLOS) — *Sewage* — The R D C has approved the construction of sewage works at Kingswood Warmley at an estimated cost of £68 000

WEST LANCASHIRE — *Sewerage* — The R D C has applied for sanction to proceed with the Sciton sewerage scheme at an estimated cost of £50 000

WESTON SUPER MARE — *Bathing Pools* — The U D C is considering the provision of a bathing pool in Glentworth Bay at an estimated cost of £35,000 and a swimming pool near Clevedon road at an estimated cost of £21,000



PROTECTING THE CORNERS of Concrete Columns

CHIPPED and broken concrete columns are not only unsightly but may also be a source of danger if the reinforcement is exposed. In the Croxson Gas Works Canteen and Laboratory Building, as in many other reinforced concrete structures,

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provide certain protection against the severest shocks and buffeting, and give a neat and permanent finish to the columns.

The Guards are simply attached to the shuttering before the latter is placed in position, when the shuttering is struck the Guards remain as an integral part of the building.

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Tenders Accepted.

ARDGLASS — *Water Supply* — The Downpatrick R D C has accepted the tender of G P Trentham, Ltd, at £2,817, for works in connection with the construction of water supply works

BRIDGWATER — *Cattle Market* — The F C has accepted the tender of H W Pollard & Sons, at £20,540, for the construction of a cattle market

BURNLEY — *Bridge* — The T C has accepted the tender of G Wimpey & Co, Ltd at £6,996, for the reconstruction of Mitre bridge, Westgate

GALLOWAY — *Water Supply* — The Galloway Water Power Co, Ltd has accepted the tender of A M Carmichael, Edinburgh, for the construction of a reinforced concrete pipe aqueduct and foundations, etc for the power station at Kendoon, the contract price is approximately £70,000

GRANGE — *Reservoir* — The U D C has accepted the tender of Birch & Sons, Grange over Sands, at £1,091 for the construction of a covered reservoir

HARWICH — *Concrete Roads* — The L C has accepted the tender of G Wimpey & Co, at £4,196, for the construction of reinforced concrete roads and incidental works in Barrack Lane and Becon Hill Avenue

HUNTER — *Water Supply* — The F C has accepted the tender of L Flaherty, Falkirk, at £9,656, for constructional works in connection with the water supply

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We can supply at short notice the following plant for reinforced concrete work —

Steel shuttering, concrete placing plant, mixers, road forms, road surfacers, batcher plants, central mixing plant, aggregate weighing equipment, bulk cement plant, adjustable shores and excavators.

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HULLION — *Swimming Bath* — The Lancashire Education Committee has accepted the tender of J Turner & Sons (Preston), Ltd, at an approximate cost of £6,000, for the construction of a swimming bath at the grammar school

KIDSGROVE — *Reservoir Cover* — The U D C has accepted the tender of S & P Cope, Smallthorne, at £210, for the construction of a reinforced concrete cover for Mow Cop reservoir

LONDON — *Subway* — The City Corporation has provisionally accepted the tender of Mitchell 1805, Sons & Co, Ltd, London at £21,066, for the construction of a subway at the junction of King William Street, Cannon Street and Gracechurch Street. Other tenders submitted Chas Brand & Son, £27,833, John Mowlem & Co, Ltd, £22,890, Kinnear, Moodie & Co, Ltd, £22,740, John Cochrane & Sons, Ltd, £22,410

LONDON (BLRMONDSLY) — *Concrete Piling* — The L C C has accepted the tender of Simplex Concrete Piles, Ltd, at £990, for pile foundations for a block of dwellings on the Dickens' estate. Other tenders submitted François Cementation Co, Ltd, £949, Holst & Co, Ltd., £950, Bierrum & Partners, £955; Concrete Piling, Ltd, £1,334; J Gill (Contractors), Ltd, £1,339, S Williams & Sons, Ltd, £1,425, Franki Compressed Pile Co, Ltd, £1,900

January, 1935



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ULTRA RAPID SETTING under absolute control.
ACTUAL GREATER DENSIFICATION of the cement.
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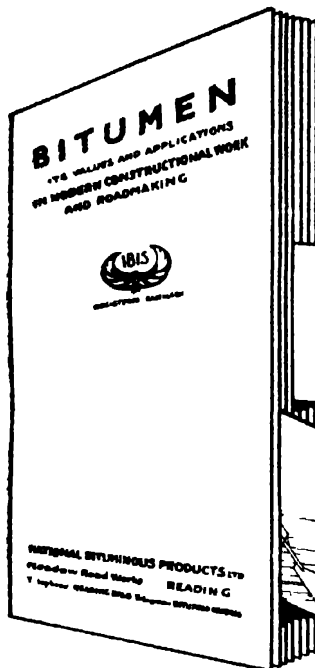
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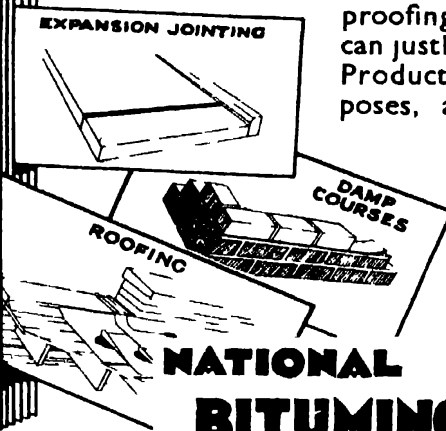
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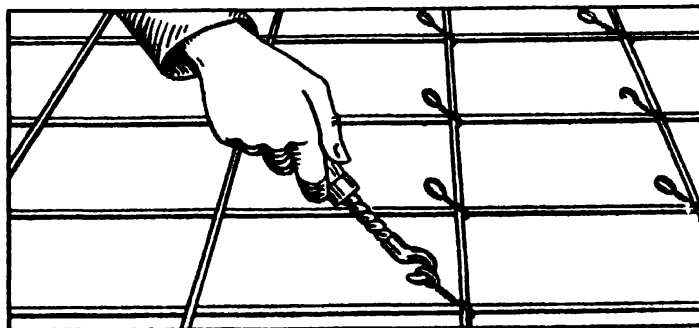
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